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HYDRO-ELECTRIC POWER STATION, CALIFORNIA GAS & ELECTRIC CORPORATION, DE SABLA, CAL.

By J. D. Galloway.*

There has recently been put in operation an electric generator at de Sabla, Cal., which, in point of capacity, general design and conditions of operation, is well worth noting. One of the generators at this station is of 5,000-KW. capacity and is run by a single water wheel operating under a static head of water of 1,528 ft. It is the third unit installed at this point, and it is the purpose of this article to describe the entire power station.

THE CALIFORNIA GAS & ELECTRIC CORPORATION.

Before entering into a description of the plant at de Sabla a short sketch of the entire system, of which it forms a part, is necessary in order to gain a clear conception of the work done (see map, Fig. 1). The California Gas & Electric Corporation is the direct outgrowth of the small plants, started between seven and eight years ago, by the Yuba Power Co. and the Nevada County Power Co. These two companies were combined and formed the Bay Counties Power Co., which had its principal plant at Colgate on the Yuba River. Under this company the power lines were built to Oakland, this transmission line being at the time a long step in advance, both in length of line, 142 miles, and pressure employed, 50,000 volts. (See Eng. News, Oct. 3, 1901.)

Shortly after the completion of the Colgate plant, the one at de Sabla was started under an allied company, known as the Valley Counties Power Co. This plant, being near the village of Nimshew, was known for a time by that name, and articles concerning it have appeared in print at various times under that caption. Before the completion of the de Sabla plant, the California Gas & Electric Corporation was formed. This was somewhat over a year ago. Among the companies included in the new organization, and not already mentioned, were the Oakland Gas, Light & Heat Co., The Sacramento Electric, Gas & Railway Co., with power station at Folsom, The Standard Electric Company of California, with power station at Electra; and the South Yuba Water Co., and the Central California Electric Co., with power stations at Alta, Auburn and Newcastle.

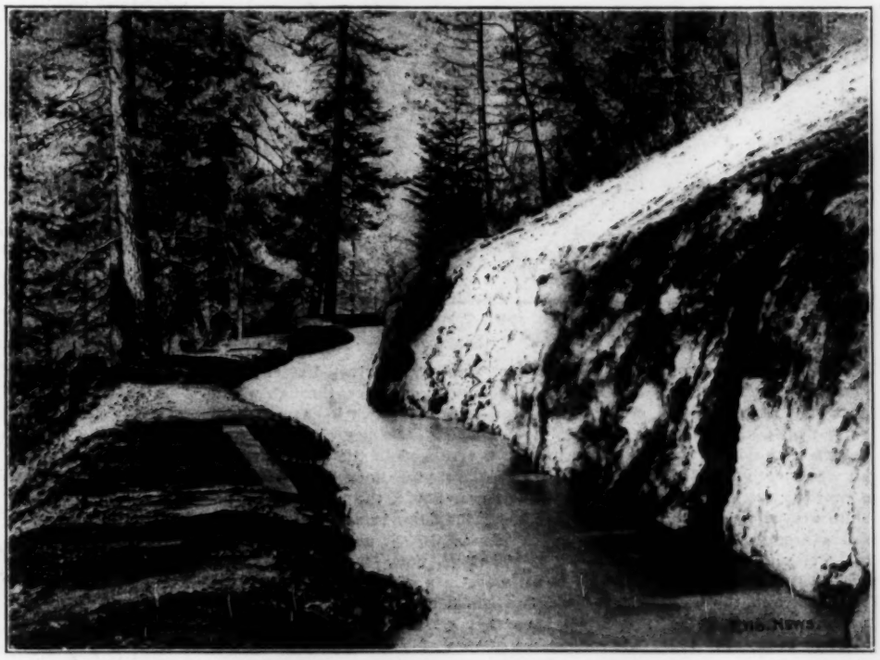
The new corporation now serves nearly a hundred cities and towns with light and power, besides street railways in Oakland, San Jose, Stockton, Chico and Sacramento, numerous suburban electric railways, cement factories, smelters, mines and mills, gold dredges, etc. It has installed over 43,000 KW. at ten different power stations in the mountains, and an addition of 10,000 KW. is now being installed at Electra.

At a risk of repetition it may be stated that until the advent of electric power generated by mountain water, California was without cheap power, the discovery of oil being made at about the time the lines to Oakland were being built. California has no coal fields, being dependent on Washington, Australia and Alaska for most of its coal. The topography of the State has rendered the generation of electric power by water a commercial possibility, and thus overcome, together with the oil supply, the deficiency in power supply.

The Sierra Nevada Mts. along the eastern border extend almost the entire length of the State. At their base, in the northern portion, lie the broad Sacramento and San Joaquin Valleys, in which are many populous towns and cities. The mountains are steep and high, those at the headwaters of the Yuba, American, Mokelumne and Tuolumne rivers ranging from 9,000 to 13,000 ft. in height. The snowfall is excessive, reaching over 30 ft. on the higher parts. The middle zone is covered with a dense growth of coniferous timber, which extends from elevation 2,000 to elevation 9,000 ft. The rainfall varies, increasing toward the north and reaching over 70 ins. at de Sabla. This combination of heavy forests and excessive snowfall maintains a summer flow of the streams not to be expected in an otherwise dry climate.

The fall of the mountain streams may be judged by the distance necessary to carry the water at the various plants. At Electra, on the Mokelumne River, a nineteen-mile ditch gives a head of 1,460 ft.; at Colgate, on the Yuba River, an eight-mile flume gives a head of 698 ft.; at de Sabla, a twelve-mile ditch gives a head of 1,528 ft.

The various power developments owe much to the miners of early days, who constructed an extensive system of ditches, reaching well up into the mountains. These ditches, repaired and enlarged, form the basis for the location of power plants. Such is the one at Electra and such is the one at de Sabla.



VIEW OF BUTTE CREEK DITCH, DE SABLA HYDRO-ELECTRIC POWER STATION.

THE DE SABLA POWER PLANT.

The de Sabla plant was started by the purchase of an unused system of mining ditches known as the Cherokee. The system takes water from the West Branch of the North Fork of the Feather River and from Butte Creek. The power house is on Butte Creek, about 21 miles from Chico. The Centerville plant of the corporation, having 1,700 KW. installed, lies upon the same stream lower down. Water from the de Sabla plant, after driving the machines, flows into the creek, and about 1,000 ft. below enters the ditch of the Centerville plant, where an eight-mile ditch gives a static head of 577 ft.

DITCHES AND FLUMES.—The main supply of water is taken from Butte Creek in a conduit twelve miles long. In addition, water is brought from the West Branch of the North Fork of the Feather River by ditches extending well up into the mountains. The amount of water available is amply sufficient for the present installation of 9,000 KW., without storage, as the streams head in the snow fields of the higher mountains.

Water is diverted from Butte Creek by a crib dam about 40 ft. high, placed in a narrow portion of the canyon. It enters the flume through

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a tunnel provided with head gates. Owing to the rocky formation of the canyon, the water is carried mostly in flumes for the first five miles, the flumes growing less frequent farther down. The lower seven miles is practically all in earth of a red clay formation, considered the best ground for such purposes.

The flumes are 4 × 6 ft. in section (Fig. 2), built of native lumber. Flumes thus built are practically water-tight. The ditch (see view on first page of this issue) is built with side slopes of 1 ft. vertical in 6 ins. horizontal in the red clay and with vertical sides in rock. These slopes stand up well in this character of ground. The grades of the ditch are not uniform, as the old ditch was merely cleaned out with such rectification of grade as was possible. In general the grade was 0.0016 with short flumes the same. Long flumes have a grade of 0.0032. Grades of 0.0024 are used in the clay at the lower end without cutting of banks. As now built, the ditch has a capacity of 65 sec. ft., and the section gives a depth of water equal to one-half the width. Discharge gates and sand boxes are provided at intervals.

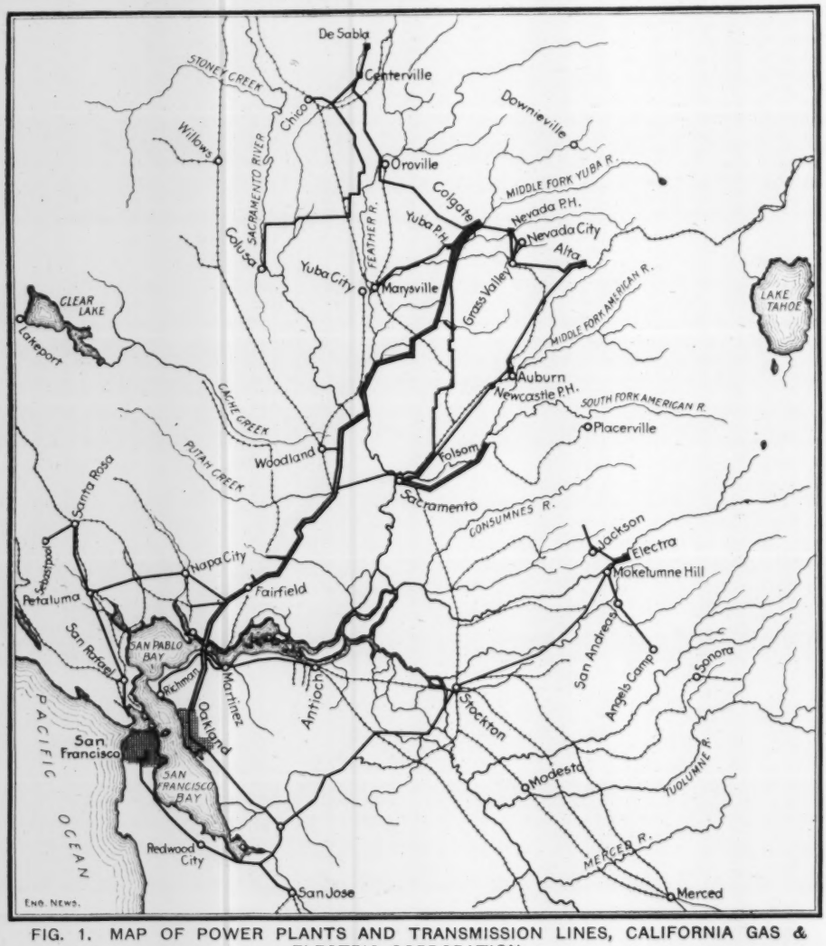

FIG. 1. MAP OF POWER PLANTS AND TRANSMISSION LINES, CALIFORNIA GAS & ELECTRIC CORPORATION.

RESERVOIR.—At the lower end of the ditch a regulating reservoir has been built, with a present capacity of 9,000,000 cu. ft. and a prospective capacity of 12,000,000 cu. ft. on completion of the dam. Besides taking up the variation in load it contains over one day's supply of water for the plant in case of accidents to the ditch.

The reservoir was built on the summit of the ridge separating the two streams that supply the water. The water from the ditch enters at the upper end, where a weir is placed for measurements. The dam is an earth fill, taken from the reservoir site. No core was formed, as the entire dam is of the red clay formation mentioned above. It was compacted by flooding sections of it with water during construction. No leak has appeared in the face since the water has been admitted, but a small amount finds its way through volcanic strata below. The dam has a height of 50 ft. and side slopes of 2½ to 1. A small spillway is provided for possible overflow from the ditch, but the reservoir has no drainage area.

The outlet gates are placed on the outer toe of the dam. Beneath the dam extend two pipes encased in concrete. These are of riveted steel plates and lead from a concrete chamber, through which water enters from the top, which is covered with a grating of iron bars. A sluice gate and pipe are also provided, arranged in the same manner.

PIPE LINES.—Two pipe lines lead from the reservoir to the power house, the total length, including that under the dam, being 6,341 ft. Line No. 1 supplies two 2,000-KW. units, and line No. 2 supplies a 5,000-KW. unit. The two lines are practically the same in profile, except that at the lower end Line No. 2 has a longer tunnel than Line No. 1, so that a description of Line No. 2 will suffice. A profile of the portion of the line in tunnel is shown by Fig. 3. From the reservoir to the "common point" the pipe follows the contour of the ground, crossing two ravines on trestles and passing through a short tunnel. From the "common point" to the power house it lies in a tunnel cut mostly in solid rock at an angle of about 34 degrees from the horizontal. This tunnel is over 1,800 ft. long, 6 ft. high and 8 ft. wide, the extra space being for a future additional pipe line. Work on the tunnel was carried on at the top and bottom and also from two side adits. Air drills were used, with a compressor run by a motor, and the tunnel was lighted by electric lights. The surveys were accurately made and the headings met without sensible error.

The alinement of the pipe is a straight line from the power house to near the [illegible] where a curve of large radius is used [illegible] turning to the dam. Three kinds of pipe [illegible] used, lap-welded steel with flanged joints [illegible] lower end; then lap-welded steel with bell [illegible] riveted, and thence riveted steel plates. [illegible] welded pipe was of 30-in. outside diameter [illegible] the riveted pipe of 30-in. inside diameter. [illegible] riveted pipe was made of sheets about 5 ft. [illegible] and came to the site in lengths of about 2[illegible] In general, the longitudinal and transverse [illegible] were double riveted. The angle about 250 ft. f[illegible] the power house was made of cast steel.

In this connection it might be mentioned [illegible] in the new installation at Electra, another of [illegible] corporation's power plants, pipe made of riv[illegible] steel plates is being used under a head of [illegible] ft., and an inside diameter at the lower end of [illegible] ins. The lap-welded pipe was furnished by [illegible] National Tube Works. The riveted pipe was f[illegible] nished by the Schaw-Batcher Co., of Sacram[illegible] who had the contract for laying pipe No. 2.

In assemblying the pipe, that above the "co[illegible] mon point" was brought to the site in wagons [illegible] rolled into place by hand. In the tunnel the [illegible] was lowered from the upper end on a car, a tra[illegible] being built through the tunnel for that purpose. A pipe was laid the entire length of the line that supplied compressed air for riveting and calk[illegible] ing. Air was also used to run the hoist at the head of the tunnel.

From the "common point" to the reservoir the pipe is laid in a trench, excepting short parts in a tunnel and on two trestles. It is anchored at points by concrete blocks. In tunnel each section rests upon a concrete block and heavier anchor blocks are placed at intervals.

The pipes are provided with stand-pipes at the gates below the dam, and with 6-in. air valves at intervals. Blow-off gates are provided at low points and manholes are cut in at convenient intervals.

GENERAL ARRANGEMENT OF POWER PLANT.—Fig. 4 is a cross-section through the building at the 5,000-KW. unit. This unit, with space for another of like size, occupies one half of the building. The two 2,000-KW. units take up the other half of the building. Fig. 5 is a general interior view of the building and equipment.

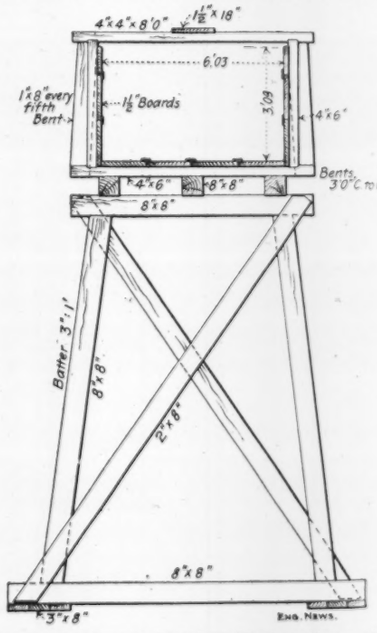

Fig. 2. Section of Standard Timber Flume, California Gas & Electric Corporation.

WATER WHEELS.—The pipe lines lead directly to the water wheels, Line No. 1 being divided by a cast steel "Y" to serve the two 2,000-KW. generators. Gates operated by water pressure are placed just back of the nozzles. The wheels (see

Fig. [illegible] were manufactured by the Abner Doble Co., of San Francisco, and each wheel is operated by a single nozzle provided with a needle valve. Regulation is secured by Lombard governors, which adjust the nozzles as the load varies. The

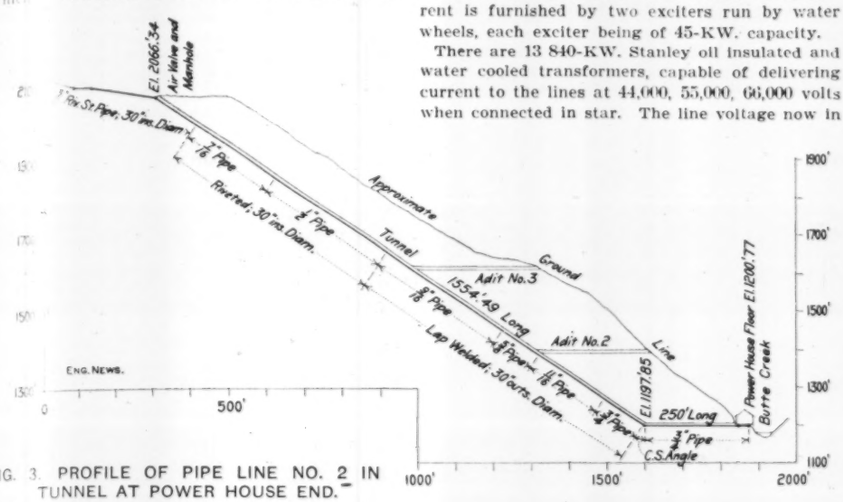

FIG. 3. PROFILE OF PIPE LINE NO. 2 IN TUNNEL AT POWER HOUSE END.

governor and nozzle are shown by Fig. 7. The water after leaving the wheels passes directly into the creek. The water wheels for the 2,000-KW. machines run at 240 revs., and for the 5,000-KW. machine at 400 revs. per min. All machines are of the two-bearing type, the water wheel overhanging. The shaft of the 5,000-KW. machine is hollow, 20 ins. in diameter at the central part and 16 ins. in diameter at the bearings. The buckets (Fig. 6) are bolted to a solid rolled steel disk, the surface of the bucket being ellipsoidal in shape.

GENERATORS.—The two 2,000-KW. generators are of the inductor type, made by the Stanley Electric Mfg. Co. The 5,000-KW. generator is of the revolving field type, made by the same company. All are 60-cycle, 3-phase, 2,300-volt machines. It is worthy of note that directly after installation the 2,000-KW. units ran over seven months without stopping and without damage.

A description of the 5,000-KW. machine appeared in Engineering News of Oct. 27, 1904. The capacity is there given as 4,000 KW., but it has been found that the generator can be run normally at 5,000 KW., with a possible overload up to 6,000 KW. for a short time. The exciting current is furnished by two exciters run by water wheels, each exciter being of 45-KW. capacity.

There are 13 840-KW. Stanley oil insulated and water cooled transformers, capable of delivering current to the lines at 44,000, 55,000, 66,000 volts when connected in star. The line voltage now in use is 55,000. The position of the transformers is shown in Fig. 4.

SWITCHES.—The high tension switches are of the oil-break type, as shown by Fig. 8. They are situated in a gallery above the transformers, the wires leading directly up from the transformers. The floor of the gallery is of terra cotta on steel beams. Each switch is placed in a compartment formed of terra cotta tiles. Each set of switches is operated by a lever from the ground floor, the lever also operating the knife switch, which is fastened to the wall of the compartment. The three bus bars are enclosed in terra cotta tile compartments running the full length of the station. The wires pass from the building through glass windows 36 ins. diameter with 6-in. openings in the glass.

BUILDING.—The building, the exterior of which may be seen in the general view, Fig. 9, is a steel frame throughout, covered with corrugated galvanized iron. The roof is double and no trouble has been had from condensed water. The foundations were carried to bed rock, the outer walls being over 30 ft. high. The foundations for the machines rest upon bed rock, and were made of rubble stone in gravel concrete. Two traveling cranes of 25 tons capacity were provided. They are hand operated and were made by the Cyclops Iron Works of San Francisco.

The compact nature of this class of power generators may be judged by the fact that when the station installation is complete, machines, generating 14,000 KW., with transformers and switches, are contained in a building 110 ft. long and 57 ft. wide.

A substantial residence of 12 rooms has been built for the accommodation of the station employees.

LINES.—The line leading from the plant is No. 6/0 aluminum cable. The country is very rough and for several miles long spans across canyons were used, running from 900 to 1,800 ft. The triangular construction was not maintained on long spans, but each wire was held on a pole frame secured by several insulators. This line has been in operation for two years without break.

Since the commencement of the de Sabla plant various engineers have been connected with the work. It was located and started under the late T. E. Theberath. The installation of the two 2,000-KW. units was completed in 1903 under Mr. L. M. Hancock, then general superintendent. The installation of pipe line No. 2 and the 5,000-KW. units were carried out in 1904 under Mr. F. G. Baum, Transmission Engineer. On this work Mr. D. H. Duncanson was superintendent of construction. Mr. James H. Wise was in charge of the civil engineering, and Mr. George H. Bragg, of the electrical engineering at the plant. The building was erected under the direction of the writer, who also acted as consulting civil engineer during the work of 1904.

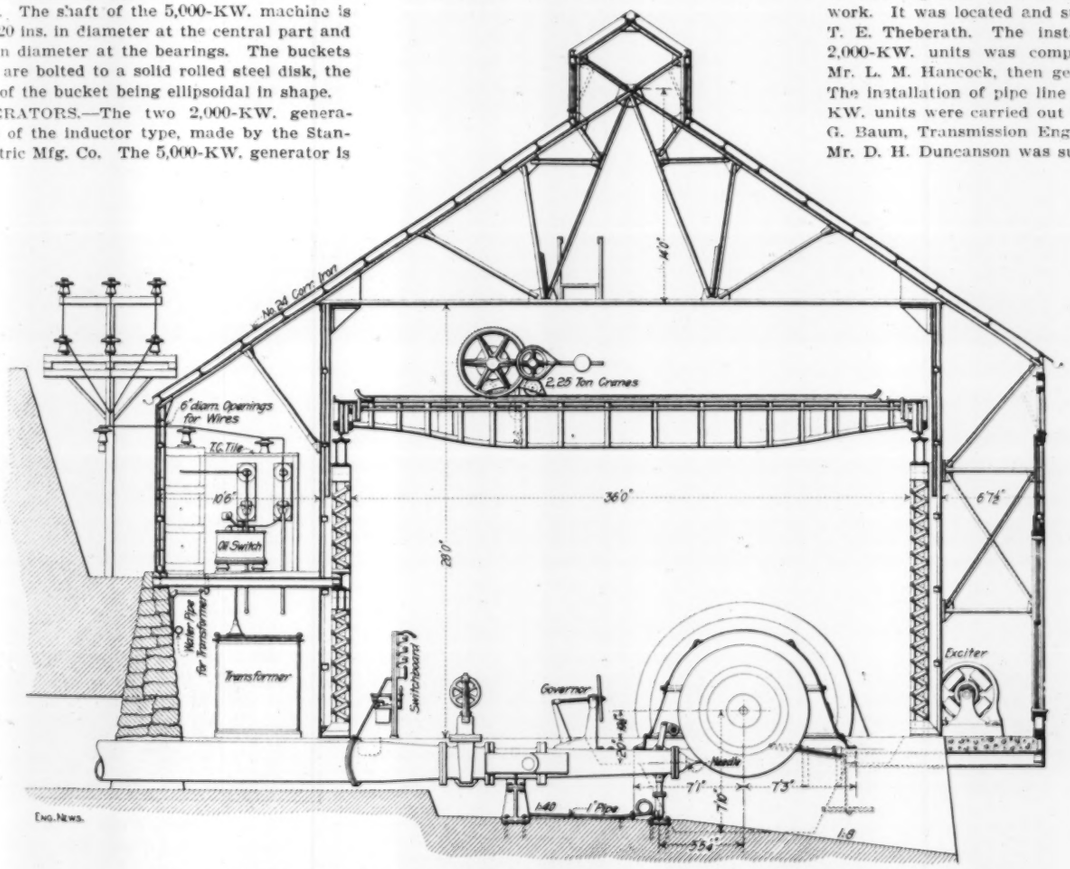

FIG. 4. CROSS-SECTION OF THE DE SABLA ELECTRIC POWER STATION AT 5,000-KW. UNIT.

THE FLOW OF AIR IN METAL PIPES.*

By J. H. Kinealy,† M. Am. Soc. Heating and Ventilating Engineers.

The expression for the friction of the flow of air in metal pipes of a circular or square cross-section may for all practical purposes be taken as

$$F = \left(\frac{v}{5{,}200}\right)^2 \frac{0.3\,l}{d}. \qquad (1)$$

In this formula, F is the friction in ounces per square inch; v, the velocity of the air in feet per minute; l, the length of the pipe in feet; and d, the diameter in inches of the pipe if round or the side if square.

If we solve 1 for v we get

$$v = 9{,}500\sqrt{\frac{d\,F}{l}}. \qquad (2)$$

If the pipe is round, its area of cross-section in square feet is

$$\frac{0.7854\,d^2}{144} = \frac{d^2}{183};$$

and if the pipe is square its area of cross-section in square feet is

$$\frac{d^2}{144}.$$

The product of the area of cross-section in square feet multiplied by the velocity of flow through the pipe in feet per minute, is equal to the cubic feet, C, of air flowing through the pipe per minute.

That is, for a round pipe,

$$C = \frac{9{,}500\,d^2}{183}\sqrt{\frac{d\,F}{l}} = 5.2\sqrt{\frac{d^5\,F}{l}}, \text{ about;} \qquad (3)$$

and for a square pipe

$$C = \frac{9{,}500\,d^2}{144}\sqrt{\frac{d\,F}{l}} = 6.6\sqrt{\frac{d^5\,F}{l}}, \text{ about.} \qquad (4)$$

These equations apply only to smooth pipes, and since it is impossible to make in practice pipes which shall be smooth everywhere inside, it is better to say

$$C = \begin{cases} 4.4\sqrt{\dfrac{d^5\,F}{l}}, & \text{for a round pipe.} \\ 5.5\sqrt{\dfrac{d^5\,F}{l}}, & \text{for a square pipe.} \end{cases} \qquad (5)$$

An inspection of (5) shows that for all practical purposes we may say that the amount of air which will flow through a round pipe of a given diameter and length, for a given value of F, is about 0.8 as much as will flow through a square pipe under the same conditions.

*A paper read at the Chicago meeting of the American Society of Heating and Ventilating Engineers.

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TABLE I.—Values of k for Various Values of $\frac{r}{d}$.

$\frac{r}{d}$	k
½	5
1	4
1½	3
2	2

A bend in the pipe tends to increase the amount of friction, and has the same effect as increasing the length of the pipe by an amount which depends upon the ratio of the radius of the middle of the bend to the diameter of the pipe, and upon the diameter of the pipe itself. If we call e this increase in length we may say

$$e = k\,d \qquad (6)$$

For a 90-degree bend where the radius of the middle of the bend is $\frac{d}{2}$, k is about 5; and for a 90-degree bend where the radius of the middle of the bend is d, k is about 4.

FIG. 5. GENERAL INTERIOR VIEW OF DE SABLA ELECTRIC POWER STATION.

Table I. gives the values of k for 90-degree bends and various values of the ratio obtained by dividing the radius, r, of the middle of the bend by the diameter, d, of the pipe if round, or the side if square.

If we find the equivalent increase of length of a pipe due to each bend, and add this to the length in feet of the pipe, we have what may be termed the equivalent length of the pipe, which is always greater than the actual length as measured. Thus, let us suppose we have a square 12-inch pipe, 100 feet long, which has one square bend and one bend for which the radius of the middle is 18 inches or 1.5 times the size of the pipe, and we wish to know the equivalent length of the pipe.

From Table I. we see that k for the square bend, $\frac{r}{d}$ equal ½, is 5; and for the other bend, where $\frac{r}{d}$ is 1.5, k is 3.

Hence for the square bend the increase in length is

$$5\,d = 60 \text{ feet;}$$

and for the other bend the increase is

$$3\,d = 36 \text{ feet.}$$

Therefore, the total equivalent length of the pipe is

$$100 + 60 + 36 = 196 \text{ feet.}$$

That is to say, the friction of this 12-inch pipe, 100 feet long and having the two bends, is the same as it would be in a straight 12-inch pipe 196 feet long.

It is evident that we may assume various values of d, F, and l in (5) and calculate the amount of air which would flow through the pipe under the assumed conditions.

Table II. gives the values of C for various values of d and F for square pipes 100 feet long, as calculated by (5).

TABLE II.

Cubic feet of air that will flow per minute through square pipes of various sizes, 100 feet long, for various values of F.

Size of Pipe in Inches.	F, or Loss by Friction in Ounces per Square Inch.						Area of Cross-Section of Pipe in Square Inches.
	0.1	0.2	0.3	0.4	0.5	0.6	
6	150	220	270	310	340	370	36
8	280	440	540	620	700	770	64
10	560	790	980	1,100	1,300	1,350	100
12	870	1,300	1,500	1,700	1,950	2,100	144
14	1,300	1,800	2,200	2,600	2,900	3,100	196
16	1,800	2,500	3,100	3,500	4,000	4,300	256
18	2,400	3,400	4,100	4,800	5,300	5,800	324
20	3,100	4,400	5,400	6,300	7,000	7,700	400
22	4,000	5,600	6,900	7,900	11,000	12,000	484
24	4,900	6,900	10,400	12,000	13,500	15,000	576
26	6,000	8,500	12,500	14,500	16,000	18,000	676
30	7,200	10,200	15,000	17,000	19,000	21,000	900
34	11,500	16,300	20,000	24,000	26,000	30,000	1,156
38	15,300	16,600	27,000	31,000	35,000	38,000	1,444
42	20,000	28,000	34,000	40,000	44,000	48,000	1,764
46	26,000	35,000	43,000	50,000	56,000	61,000	2,116
50	31,000	43,000	53,000	62,000	69,000	75,000	2,500

Table II. is made for square pipes, but it can be used for rectangular pipes that are not square and for round pipes. The table may be used for any rectangular pipe whose longest side is not more than twice the shortest. Thus the friction in a rectangular pipe 7 by 14 inches is only about 5 per cent. more than that in a square pipe 10 by 10 inches of the same length. As long as the shortest side of the cross-section of the pipe is not less than one-half the length of the long side, the friction of a rectangular pipe will not be more than 5 per cent. greater than the friction in a square pipe of the same cross-section and the same length; and a difference of 5 per cent. is too small to be considered in such work as laying out a system of pipes for a heating or ventilating system.

Table II. may be used for round pipes if we assume that the number of cubic feet of air carried by a round pipe of a given diameter and length is 0.8 of the number of cubic feet of air carried by a square pipe of the same length whose side is equal to the diameter of the round pipe. Thus the table shows that the friction of 1,300 cubic feet of air passing per minute through a square pipe 10 by 10 inches and 100 feet long is 0.4 of an ounce. A round pipe 10 inches in diameter and 100 feet long would have the same friction if 880, equal 0.8 of 1,100, cubic feet of air passed through it per minute.

Table II. may be used to determine the size of pipe required to carry a given amount of air per minute with a given loss by friction. This may best be made clear by examples.

Example I. Determine the size of a rectangular pipe required to deliver 3,000 cubic feet of air per minute a distance of 100 feet with a loss by friction of 0.2 of an ounce.

Looking under the column headed 0.2, we find opposite 16 in the first column, 2,500; and opposite 18 in the first column we find 3,400. That is, a 16-inch square pipe will deliver 2,500 cubic feet of air per minute a distance of 100 feet with a loss by friction of 0.2 of an ounce, and an 18-inch square pipe will deliver 3,400 cubic feet of air per minute the same distance with the same loss. Since 3,000 is about midway between 2,500 and 3,400 it is evident that we need a 17-inch square pipe, since it is midway, almost at least, between a 16-inch and an 18-inch pipe.

If we do not want to use a square pipe we proceed as follows: The area of a 16-inch square pipe which will deliver 2,500 cubic feet per minute is, from the table, 256 square inches, and the area of an 18-inch pipe which will deliver 3,400 cubic feet per minute, is 324 square inches.

The mean of these is 290, equal $\frac{256 \times 324}{2}$, square inches.

That is, the pipe would have an area of 290 square inches to deliver 3,000 cubic feet of air 100 feet with a loss by friction of 0.2 of an ounce. We may use a pipe 12 by 24; or 15 by 20; or 16 by 18 inches.

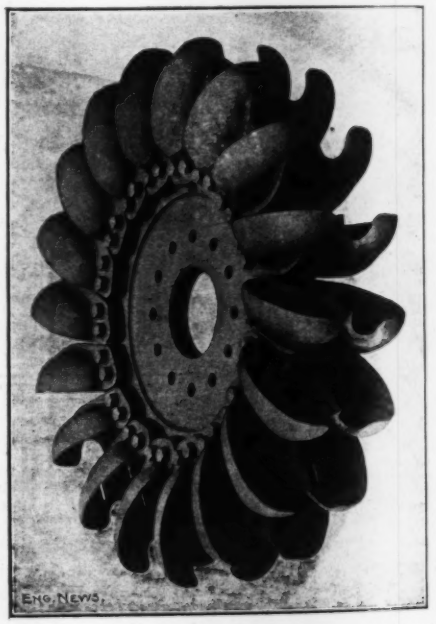


Fig. 6. View of Doble Water Wheel for 5,000-KW. Electric Unit, de Sabla Power Station.

Example 2. Determine the diameter of a round pipe that will deliver 10,000 cubic feet of air per minute a distance of 100 feet with a loss by friction of 0.5 of an ounce.

Since, as has been pointed out, a round pipe will deliver only 0.8 as much air as a square pipe of the same size with the same loss by friction, if we divide 10,000 by 0.8 we get the amount of air which a square pipe

of the ... size as the required round pipe will deliver. And 1... ivided by 0.8 is 12,500.

Look... nder the column headed 0.5, we find that a 24-inch ... pe will deliver 13,000 cubic feet of air per mi... with a loss by friction of 0.5 of an ounce. Hence ... nch round pipe is what we shall use.

Exa... 3. Determine the size of a square pipe required ... deliver 1,500 cubic feet of air per minute a distance of 300 feet with a loss by friction of only 0.1 of an ounce.

Delivering 1,500 cubic feet of air 300 feet is equivalent to delivering at a distance of 100 feet an amount of air equal to 1,500 multiplied by the square root of the quotient obtained by dividing 300 by 100. The quotient obtained by dividing 300 by 100 is 3; and the square root of 3 is 1.7.

Hence the pipe which will deliver 1,500 cubic feet of air a distance of 300 feet will deliver 2,600, equal 1.7 multiplied by 1,500, cubic feet of air 100 feet with the same loss by friction.

Therefore, we proceed as before, and find the square pipe which will deliver 2,600 cubic feet of air per minute a distance of 100 feet with a loss of 0.1 of an ounce. This we find must be a 19-inch square pipe, although a pipe 18 by 19 inches will be large enough.

Example 4. Determine the size of pipe 150 feet long with one square 90-degree bend and two curved 90-degree bends, required to deliver 2,000 cubic feet of air per minute with a loss by friction of 0.3 of an ounce.

In working such an example as this it is usual to assume first the probable increase in the amount of air to compensate for the length and bends, and by practice one soon learns to assume about the proper increase. Here, however, we shall assume no increase at all and proceed as follows:

Delivering 2,000 cubic feet of air 150 feet is equivalent to delivering 2,500, equal 2,000 multiplied by the square root of the quotient obtained by dividing 150 by 100, cubic feet of air 100 feet. And the table shows that for a loss 0.3 of an ounce a 15-inch pipe will be required. We may assume that for the square bend k, given in Table I., is 5; and for each of the round bends k is 3. Hence the equivalent length due to the bends is

$$5 \times 15 + 2 \times 3 \times 15 = 75 + 90 = 165 \text{ feet.}$$

The equivalent length of the pipe, therefore, is

$$150 + 165 = 315 \text{ feet.}$$

That is, because of the bends we must deliver the air not 150 feet, but 315 feet. And delivering 2,000 cubic feet of air 315 feet is equivalent to delivering 3,600, equal

$$2{,}000 \times \sqrt{\frac{315}{100}} \text{ cubic feet of air 100 feet.}$$

From Table II. we see that a 17-inch pipe will be required to deliver 3,600 cubic feet of air a distance of 100 feet with a loss by friction of 0.3 of an ounce.

But the equivalent length due to the bends in the 17-inch is

$$5 \times 17 + 2 \times 3 \times 17 = 85 + 102 = 187$$

instead of 165 as assumed.

And the equivalent length of the pipe is

$$150 + 187 = 337 \text{ feet.}$$

And delivering 2,000 cubic feet of air 337 feet is the same as delivering 3,000 equal $2{,}000 \sqrt{\frac{337}{100}}$ cubic feet a distance of 100 feet.

Table II. shows that a 17-inch pipe is not quite large enough, but nearly so, to deliver 3,700 cubic feet of air per minute a distance of 100 feet with a loss by friction of 0.3 of an ounce. A pipe 17 by 18 would be amply large.

Table II. shows very clearly how and why it is that the friction of the flow of air in a round pipe is different from the friction of the flow of air in a square pipe. Thus, suppose we wish to determine the size of a square pipe and also the size of a round pipe capable of delivering 2,000 cubic feet of air a distance of 100 feet with a loss by friction of 0.2 of an ounce.

Turning to Table II., we see that a 14-inch square pipe will deliver 1,800 cubic feet of air a distance of 100 feet with a loss by friction of 0.2 of an ounce, while a 16-inch square pipe will deliver 2,500 cubic feet of air the same distance with the same loss. That is, the 14-inch pipe is slightly too small and the 16-inch pipe is too large. We should use, therefore, a 15-inch square pipe.

To find the round pipe required, we divide 2,000 by 0.8 and get 2,500. Then we look in Table II. and find the size of pipe required, and this will be the size of round pipe required to deliver 2,000 cubic feet of air a distance of 100 feet with a loss of 0.2 of an ounce by friction. From the table we see that an 18-inch pipe will be required. That is, an 18-inch square pipe will deliver 2,500 cubic feet of air a distance of 100 feet with a loss by friction of 0.2 of an ounce, while an 18-inch round pipe will deliver 2,000 cubic feet of air a distance of 100 feet with a loss by friction of 0.2 of an ounce.

In other words, the round pipe must be 18 inches in diameter, while the square pipe need be but 15 inches on the side. The round pipe must be 20 per cent. larger than the square pipe in order to deliver the same quantity of air with the same loss by friction. A round pipe of the same size as a square pipe of a given length will have just exactly the same loss by friction as the square pipe and will deliver only about 0.8 as much air as the square pipe. Further, the round pipe will take up just as much room or space as the square pipe.

A square pipe is better than a round pipe, because it will deliver more air for the same loss by friction and will take up no more space. True the square pipe will contain more metal than the round pipe, whose diameter is equal to the side of the square pipe, and the square pipe will cost more to make.

The formulas and table given here may be used to lay out all kinds of systems of piping for air, and I hope that the members of the society will find them as useful and convenient in their work as I have.

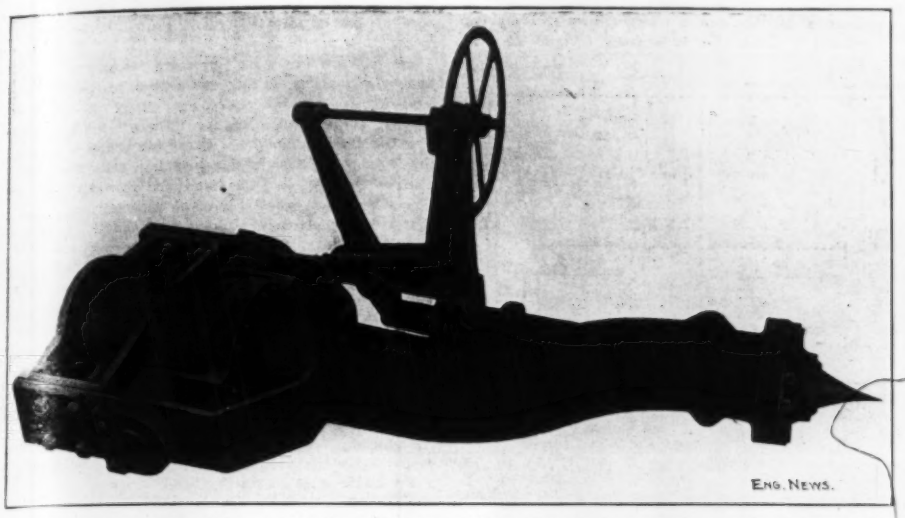


FIG. 7. VIEW OF LOMBARD GOVERNOR AND DOBLE NOZZLE FOR 5,000-KW. GENERATOR UNIT, DE SABLA POWER STATION.

MALARIA AND YELLOW FEVER.*

By Major Ronald Ross,† C. B., F. R. C. S., F. R. S., late I. M. S.

Malarial fever is, perhaps, the most troublesome pest that afflicts humanity. Fortunately, it is not a very fatal malady; but its total prevalence is so enormous that the addition which it makes to the death-roll may possibly exceed that of tuberculosis, cholera, plague and yellow fever put together. Indian statistics show that one-third of the admissions into hospital among the troops are due to malaria; and in many localities in Africa it has been proved that nearly all the native children are infected. Indeed, malaria takes the place in the tropics of chest complaints and rheumatism in temperate climates—where it does not kill, it frequently cripples, or predisposes to other diseases. Moreover, the fact that it tends to abound most in the most fertile countries renders it above all the enemy of civilization—the great bar of progress in the tropics. I have often thought that Africa has so long resisted the civilization of Europe chiefly because of its malaria which strikes down the missionary, the pioneer, the soldier, the official, the merchant, who comes to open it up. The growth of many of the most promising colonies has undoubtedly been stunted by it. We should try to imagine what the world might have been without malaria—the tropical and subtropical regions would probably have been as far advanced as the temperate countries now are.

The history of our knowledge of the disease is briefly as follows. The Romans were not only cognizant of it, but also, apparently, of the fact that it is connected with stagnant terrestrial water, and can be reduced by drainage. In the eighteenth century the discovery of the specific action of cinchona bark (from which quinine is made) enabled Torti to discriminate the exact symptoms, especially the peculiar periodic fever; and about the same time Lancisi further emphasized the connection between the fever and stagnant water; but it was not until 1880 that the specific cause of the disease—a minute amœbic parasite living in the blood-corpuscles—was discovered by Laveran, a surgeon of the French army service in Algeria. This discovery was slowly confirmed by French,

FIG. 9. GENERAL VIEW OF DE SABLA POWER STATION, CALIFORNIA GAS & ELECTRIC CORPORATION.

*Abstracted from a paper in the initial number of "The University Review" (British). The original paper was entitled "Malaria and a Moral." Reference to this abstract and to the present outbreak of yellow fever at New Orleans may be found on our editorial pages.—Ed.

†Sir Alfred Jones Professor of Tropical Medicine in the University of Liverpool, England.

Italian, American and Russian observers, and by one Briton, Vandyke Carter; but the problem as to how the parasites effect an entry into our bodies—that is the problem a solution of which was essential for the purposes of prevention—resisted all such efforts as were made, until it was resolved by a long series of researches* carried out between 1895 and 1899. It was then shown that the parasites pass a portion of their life-history in certain kinds of mosquitoes—the insects become infected by biting infected persons, and then, after the lapse of a week or more, inoculate them into healthy human beings. Every stage in the life of the parasites was carefully mapped out, described and figured by many observers, and healthy birds and human beings were infected under stringent experimental conditions by infected mosquitoes. The long-known connection between malarial fever and terrestrial water was explained by the fact that the malaria-bearing species of mosquito breed in such waters; and in fact all that now remained to be done was to apply the discovery on a large scale to the saving of human life. But though more than five years have elapsed since such a course became practicable, and though decisively successful experiments have been made in a few isolated towns, we still look in vain for any comprehensive or business-like treatment of the subject by those excellent people who are supposed to manage this country and its dependencies.

One of the first questions suggested by this history is why, in the name of reason, was not the old method of drainage more frequently employed against the disease? So far as I can ascertain, the principle was known to the Romans—it has certainly been a sanitary dogma for centuries. Long before the phenomenon was explained by the mosquito-theory, men knew that malaria is connected with stagnant water; and the good effect of drainage is constantly referred to in old medical works. I well remember the prominence which was given to the subject in the lectures delivered to army surgeons at Netley in 1881, and also in certain official proposals for reducing malaria in India published a few years later. But only isolated attempts seem ever to have been made. Nearly every tropical town which I have visited abounded in stagnant ditches, ponds, puddles, rice-fields and even marshes. When I first saw the old colony of Sierra Leone in 1899—a colony nick-named "the white man's grave," because of its malaria—Freetown, its capital, was full of such waters, of which many were certainly made, not by nature, *but by unskilful engineering* (italics our own.—Ed.) Heaven was blamed for the disease. The authorities were not at fault, of course—the "climate" was pestiferous. Yet, if those authorities had chosen to spend a few hundred pounds year by year in removing waste water in accordance with the dogma referred to, thousands upon thousands of lives would have been spared, and the localities would probably have become many times more prosperous than they now are. As for the expense, we must remember that the invaliding of a single soldier costs considerably over a hundred pounds.

The history of the investigations which have revealed the cause of malaria affords many lessons, pleasant and unpleasant. It is a curious fact that Laveran's discovery (1880—Ed.) of the "germ," although it enabled medical men to diagnose the disease with certainty and also to render their medication more exact, remained almost unused in medical practice for nearly twenty years. No effort was made by the heads of our public medical services to enforce attention to it. Until quite recently, indeed, doctors in the tropics were not even provided with microscopes.† Another curious fact is that on several occasions the investigations, far from receiving general assistance, were actually impeded by "professional brethren." On one occasion Drs. Plehn and Rivenburg and myself were practically turned out of a hospital in Calcutta—whither we had gone in order to find cases of malaria suitable for study—and, be it understood, without the shadow of a reason.

My readers may think that such stupidity is almost inconceivable—so it is, but it is very common, and, still worse, the perpetrators almost always flourish, and continue to flourish, like the green bay-tree, without ever being cut down, and the nation which allows such a thing still continues to think that it is going to hold the empire of the world forever against all comers. The truth is that scientific distinction, and even knowledge, are little considered in the selection of officers for administrative posts in scientific corps. Indeed, it is commonly thought that second-rate men are chosen precisely because they are less likely to urge reforms upon their superiors.

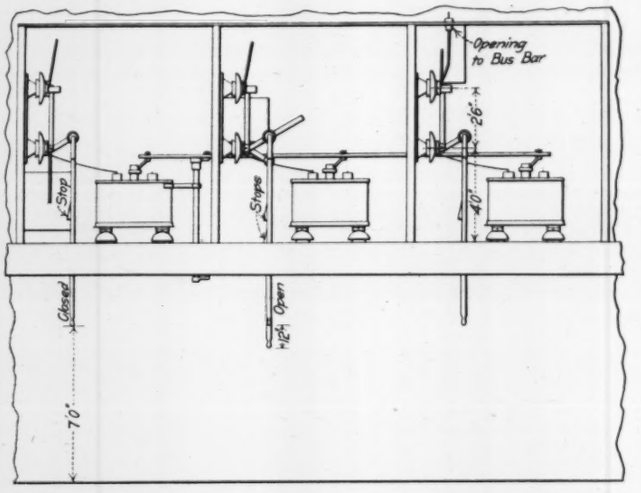
I must confess to the fact that my own experiences in research have left me with some most unchristian grudges —which I keep, carefully wrapped in cotton wool in a warm place in my soul, ready for use at the proper moment. My excuse is that the grudges are not on my own behalf, but on behalf of science and humanity. What an amazing thing it is that the last men to receive assistance and the first men to be obstructed, are precisely those who of all men are doing their utmost to benefit others, with the least chance of profit to themselves.

As soon as mosquitoes were definitely proved to be the carrying agents of malaria it became necessary, in the interests of sanitation, to propagate the information as widely as possible among the general public; and this led to many incidents which would be amusing, did they not demonstrate the extreme popular ignorance of science and scientific methods. For years the press was flooded with letters from people who seemed to resent as a personal injury the refutation of the old hypothesis that malaria is caused by a miasma rising from marshes. They could not be made to see that the old hypothesis was, after all, not far from the truth, but that it is the mosquito, the carrier of the germ, and not the germ itself, which rises from the marsh.

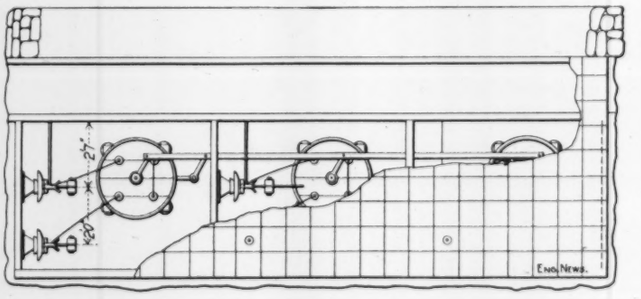
The public cannot understand that while questions of treatment are, in the present state of knowledge, largely matters of opinion, questions of pathology are on the other hand often capable of as precise a determination as questions of physics. It is most unfortunate that an absurd and antiquated name as Medicine should continue to be given to such a large science as that which deals with the nature of disease—the oldest and the most important of sciences. As well might the name Alchemy for modern Chemistry. Nature in the street connects the name Medicine with drugs but salts and senna, and attaches as little importance to a statement that malaria is carried by mosquitoes as it does to his doctor's diatribes against eating too much meat.

But perhaps the most important lesson in the history of malaria, is that our general methods of administration are somewhat obsolete. They are not those of science, energy and discipline. There is no reason why the disease should not have been nearly stamped out at this time from most of the principle malarial districts in our possessions. In 1890, immediately after the mosquito-theory was established, I advised earnestly and repeatedly that every malarious municipality should at once take steps to drain or fill up mosquito-breeding waters within its limits—at least as quickly as means allowed. Now I can say without vanity, that my advice would at least be worthy of consideration, because at that time I was certainly the only man who possessed a really intimate personal knowledge of malaria and mosquitoes; and without I had had an opportunity of becoming very well acquainted with practical tropical sanitation in general. Moreover, the measure which I advocated was merely the old one of drainage, rendered more precise and practical by addenda. Now let it be remembered that the British and colonial taxpayer supports a large and well-paid sanitary staff, which exists in almost every town of note for the special purpose of saving human life from disease by acting on the dictates of science. This staff could easily have been everywhere set in motion by a little persuasion from the head government offices, and in course of time and generally with little more expenditure than the ordinary sanitary budget could afford, waste waters would have been gradually removed and the local health greatly improved. But no efforts of ours availed. I paid several visits to Africa, and even undertook a kind of practical exposition of the subject in Sierra Leone*; but, though such a man as Sir William MacGregor has warmly taken up the scheme, though the French obtained their remarkable victory at Ismailia, and some good work has been done or attempted at Khartoum, in the Federated Malay States, Hong Kong, Madras, the Andamans, and elsewhere, I fear it must be said that nothing approaching adequate general action has been attempted by my countrymen. I do not know whose fault this is. My advice was never asked for, and certainly seldom followed. The head government offices referred to above always express themselves as being most willing to forward the work; but, if so, they are singularly powerless.

The Americans have shown us a very different picture. In 1901 they proved that yellow-fever also is carried by mosquitoes; but instead of spending six years talking about the matter, they set to work at once and cleared out yellow-fever, malaria, and mosquitoes together from Havana, and are now engaged in doing similar work along the route of the Panama Canal. Recently I had an opportunity of watching their methods there—much better methods than ours. We as a nation seem to have lost the power of prompt and resolute action; we ponder too much; are too much ruled by vacillating committees consisting of partially informed gentlemen anxious to get home to tea; are too conceited to take the advice of the men of experience; are quite above getting to the bottom of the business; and, above all things, dislike and despise anything of practical utility. If we do not take care we shall follow Spain. At least, that is the moral which the humble writer of these pages has drawn from his experiences in connection with malaria.



Front Elevation of Switch Gallery Showing Arrangement of Oil and Disconnecting Switches. (Front of Switch Room Removed.)



Plan of Switch Gallery Showing Set of Three Oil Switches in Position. (Cover partly Removed.)

FIG. 8. PLAN AND ELEVATION OF OIL SWITCH GALLERY, DE SABLA POWER STATION.

TO LAY A BRICK SEWER IN QUICKSAND the following expedient was used in St. Louis, as described by Mr. Curtis Hill in Trans. Assoc. C. E., Cornell University, Vol. XIII. An egg-shaped sewer, 4 × 6 ft., was laid for 3,800 ft. in a trench, the lower 3 ft. of which were in quicksand. The sewer was sustained in this bed of quicksand by placing burlap sacks loosely filled with dry concrete in the bottom of the trench, thereby making a concrete foundation. The description continues:

The trench was excavated until quicksand was found when sheeting was first driven along the sides, then across the line of sewer to below subgrade, thus boxing off a section of quicksand. This was excavated as rapidly as possible and the sacks of concrete were placed in immediately and tramped into the sand. These sacks were placed slightly lapping one another, the outside ones resting upon the sheeting at the sides and forming somewhat the shape of the sewer invert. The sacks were loosely filled that they might be more easily moulded and pressed into form and position. Seep water soaked and quicksand saturated the concrete and as soon as it was set the bottom could be plastered into shape for the brick work. Nine or ten inches of concrete placed in this manner gave a hard, stable foundation.

*By the author of the paper.—Ed.

†I have just heard that this remark still applies to one part, at least, of India.

*In association with other members of the Liverpool School of Tropical Medicine.

THE BARRIENTOS TUNNEL, MEXICO.

By C. Gordon Paterson.*

The Barrientos Tunnel, located about 18 kilometers (11.2 miles) from the City of Mexico, is the only double-track railway tunnel in the Republic, and is owned and operated jointly by the Mexican National and the Mexican Central Railroads. It takes its name from the Surto de Barrientos,

FIG. 1. APPROACH CUT, BARRIENTOS TUNNEL, MEXICO.

which it penetrates in a low range of hills adjoining the Valley of Mexico on the north, about midway between Mexico City and the famous Nochistongo Cut made by the early Spaniards in an attempt to pierce the hills with a canal. The object in building this tunnel was to enable the railroads to haul heavier trains, by reducing the grade from 3.5% to 0.5%, and at the same time avoiding some sharp curves between Tlanepantla and Lecheria, where the Central's short line branches to Pachuca. In the near future the tunnel will form a part of the direct line to Tampico.

The Barrientos bore is unique in four respects. First, it was driven at the fastest rate ever attained in railway tunnel work in Mexico; second, its cost per unit of excavation was lower than that of any other similar work in the Republic; third, as stated above, it is the only double-track tunnel in Mexico; and, fourth, it was completed without the loss of a single life. These remarkable features attached a peculiar interest to the work, and the methods and machines by which it was carried on.

At the north end the approach to the tunnel is through an open cut about 350 meters (1,148 ft.) long, reaching at the portal a depth of 26 meters (85 ft. 3 ins.), made mostly in rock. The tunnel itself is 224 meters (735 ft.) in length, penetrating the full distance through a hard-gained porphyry broken with clay seams which necessitated heavy timbering. The bore has a cross-section 11.30 meters (37 ft.) in width by 8.58 meters (28 ft. 2 ins.) in height. It is lined throughout with concrete blocks, those at the portal being 28 and 31 ins. by 60 ins. deep by 92 ins. long. At the southern end the portal is approached through an open cut (Fig. 1) in a tough "tepetate," about 575 meters (1,896 ft.) long and having a depth at the portal of 27 meters (89 ft.).

The method of drive was that of "heading and bench." The top heading was 2½ × 2½ meters (8.2 × 8.2 ft.), and was at the center of the top cut. Two Ingersoll-Sergeant D24 air drills mounted on columns were used in the heading; their rate was six 12-ft. holes for the cut and two 10-ft. holes for the sides drilled in six hours. A third drill of the same size and type carried forward the full-size section of the tunnel above spring line, putting in from four to six 10-ft. holes per shift. All the holes with the exception of the "cut" holes were fired by an electric battery.

Owing to the fact that dump cars could not get up to the tunnel until 15 days after work had begun, the heading and top cut had this much of a start over the bench cut, and the latter never recovered its lost ground. The muck from the heading cut was run out in small cars over a track laid on the bench and carried out beyond the bench face on timber frames at the same level; at this point the small cars were dumped into larger cars beneath. Muck from the bench face was loaded direct into these larger cars, which were hauled by mules to the portal (Fig 2) and hoisted on skips from the cut, being then run out to the dump some distance away. The cars had a capacity of three cu. meters (3.9 cu. yds.). The long hauls at each end of the bore were 1,100 and 900 meters (3,600 and 2,950 ft.).

The power plant for the tunnel included a 65-HP. boiler, which supplied both the air compressor and the derrick for hoisting the skips; and an Ingersoll-Sergeant straight line "Class A" steam-driven compressor, having a piston displacement of 628 cu. ft. of free air per minute. No repairs were necessary on this machine in the course of the work.

FIG. 2. FORTAL OF BARRIENTOS TUNNEL, SHOWING DUMPING OF MUCK CARS FROM THE HEADING INTO CARS BENEATH.

The actual time required in driving the heading through was 60 days for the 735 ft.; the total excavation of the tunnel cross-section was completed in 92 days. This includes time lost in the numerous holidays and feast days, and the weekly intermissions demanded by the laborers who quit at 3 P. M. Saturdays and returned to work only at noon on Monday. Owing to delay in finishing the approach to the southern portal, 81% of the tunnel was driven from the north end. A fall of about 700 cu. meters (915 cu. yds.) of roof in the heading caused an added delay of about 5 days. The average consumption of explosive was 96 lbs. of 60% dynamite per lineal meter (29.3 lbs. per lin. ft.).

Six drills were used most of the time and all the drill runners were Mexicans. The wages paid labor were: drill runners, $2.50 to $3.00 per shift; drill helpers, $1.00 per shift; hand drillers, $1.00 per shift; muckers, $0.60 per shift. Two blacksmiths sharpened all steels at a cost of $6.00 per day. All these values are in Mexican currency.

The contract figures on this work were as follows, all stated in Mexican currency and exchange being quoted at the time at 247%: Rock in the tunnel proper, $6.00 per cubic meter ($4.58 per cu. yd.); rock outside of tunnel, in open cut, $1.25 per cubic meter ($0.956 per cu. yd.); hard earth or "tepetate," 31 cts. per cubic meter ($0.237 per cu. yd.); loose earth, 20 cts. per cubic meter ($0.153 per cu. yd.).

Details of the work at various stages are seen in the illustrations accompanying. The tunnel is interesting to American engineers as showing the costs and rates which prevail in the sister republic. It also preaches an eloquent sermon on the value of the machine drill in heavy tunnel work.

*Contractor, 32 Calle, Colon 1012, Mexico City.

A LARGE HYDRAULIC TESTING MACHINE FOR UNIFORM LOADS.*

By R. A. Cummings.†

The following is a brief description of a testing machine and its method of application. It was especially designed and constructed by me during 1903, for the purpose of investigating shearing stresses in beams of reinforced concrete. These beams were freely supported and of variable spans. A uniformly distributed load was applied by this testing machine, the maximum being 500 lbs. per lin. ft. Many mechanical problems presented themselves in connection with the design and construction of this novel apparatus, partly on account of the temporary nature of its employment, but mainly because of the necessity for the utmost precision in the recorded results.

Hydraulic power was selected as the most practical for obtaining the large load requisite for the tests. It was decided that the best method for the distribution of the applied load was by means of a series of beams arranged as shown in the accompanying illustrations. Short lengths of I-beams were placed along the specimen beam, the points of application of the several equal loads being 12 ins. apart longitudinally. Above the shorter lengths of I-beams were arranged longer beams each one resting upon two of the shorter and above these

*A paper presented before the American Society for Testing Materials at Atlantic City, N. J., June, 1905.
†Consulting and Contracting Engineer, Pittsburg, Pa.

again were still larger beams similarly arranged. This method afforded an absolutely uniform distribution of load upon the specimen under test.

The various specimens tested were 10 × 18 ins. in cross section, the spans varying from 8 ft. to 20 ft.

It was assumed that the maximum deflection producing failure of the specimen would not exceed 2 ins. A large-scale diagram of the distribution beams was made to show the distortion of the loading with 2-in. deflection at the center. The curve of assumed deflections of specimen was plotted as a parabola, and the outlines of the lower beams laid off at right angles to a tangent to the parabola at the points of application of the loads. The maximum horizontal movement was found to be in the upper beam and amounted to one-half of the assumed deflection of the specimens. The movements of the lower tier and hydraulic jack were inappreciable. This diagram was found quite useful in designing the details.

Numerous tests were carried out by means of this machine at the works of the author under his personal supervision. Every available convenience required for investigation was utilized. The whole apparatus was "home-made" with the exception of the excellent hydraulic apparatus which was generously loaned by Mr. F. H. Stillman of the Watson-Stillman Co., manufacturers of hydraulic appliances in New York, to whom the author desires to record his obligations and thanks.

The operation of the apparatus required two observers, a recorder, and two handy-men, and the use of a steam derrick for moving the beams. When making a test, the beam was placed in position, with the observers stationed on either side to read the extensometers and deflectometer. They announced the observed action of the beam to the recorder who entered the same on the prepared blanks. One handy-man operated the hydraulic force pump and another applied the weights as required on the spinning accumulator gage. The application of load could thus be made at any rate of speed or repeated as often as needed. This feature of a hydraulic apparatus is especially useful for obtaining permanent set and the effect of speed in loading.

The spinning accumulator gage was calibrated, and checked with a dial gage for pressure up to 1,000 lbs. per sq. in. For pressure above 1,000 lbs. and up to 5,000 lbs. per sq. in., the maximum capacity of the machine, the spinning accumulator gage was used alone. A light oil instead of water was used for the hydraulic apparatus to avoid danger from freezing.

On the completion of a test the handy-men replaced the tested specimen with an untested one. It required about one-half day to make a complete test of each specimen and to place another in position; usually two tests were completed in one day.

The accompanying drawings, Figs. 1 to 6, show the machine in complete detail.

In calibrating the machine to get the actual applied load, two weighing machines having recording scales were adjusted by means of a beam to take the weight at points of application of the load. Several tests were made. Some of these were with 50-lb. increments per half-minute, and others with 100-lb. increments at 1 and 2 minute intervals. These tests were taken up to 5,000 lbs., which was the limit of the machine. The actual weight shown by the scales at each interval was recorded. Upon reaching the limit of the machine the load was released in a manner similar to its application. One of the purposes of these tests was to discover whether there was any material difference in applying the load at differing rates of speed. As a result it was determined that increments of 100 lbs. per minute were most convenient, and secured the most uniform results.

The plotting upon the curve sheets shows that in every test, upon running up the load, the load by gage exceeded the weight on the scales, and in releasing this was reversed, probably due to internal friction of the hydraulic jacks. When the load was retained at 5,000 lbs. for 8 minutes there was a falling off of pressure.

In testing specimens the webs of the lower tier of I-beams buckled, due to stretching of the spring-steel fulcrums. To guard against this we have adopted double I-beams.

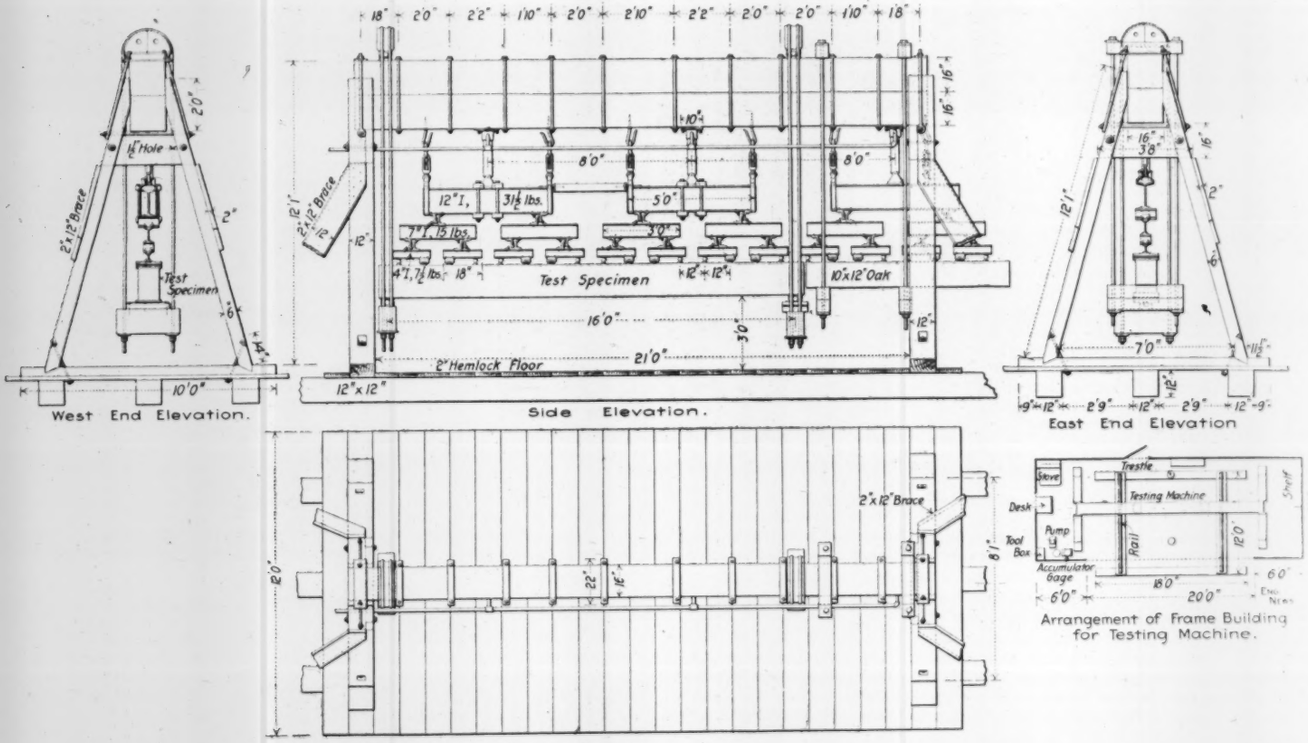


FIG. 1. GENERAL DRAWINGS OF CUMMINGS' UNIFORM LOAD BEAM-TESTING MACHINE.

CONSTRUCTION OF COFFERDAMS.*

By Thomas P. Roberts.†

There has been little advance in the methods of framing cofferdams, so far as wood structures go, in the last 20 or 30 years.

Very deep structures are rarely attempted nowadays in wood construction, preference being given to pneumatic processes for pier foundations in such cases. As to the limit in depth where the pneumatic system will prove safer and better, much will depend upon the final load expected to be carried, character of material to be penetrated, and to funds available for doing the work. This depth is sometimes taken to be 30 ft., though in the case of the Edward VII. bridge over the Thames at Kew, completed last year, open construction with timber was adopted for depths of 35 ft.

With steel piles on the interlocking system, there is no limit in depth in open construction, where due attention is given to the bracing, excepting the limit imposed by the materials encountered in the driving operations. Their use in many cases will, no doubt, obviate the necessity of resorting to expensive pneumatic processes.

It has not been my fortune to have any experience as far with steel piling, except to listen to the glowing accounts of the sales agents as they set up their models on my office table.

No two jobs are ever alike, the last one being always the meanest. If it is a leak through a rock fissure, beyond the capacity of boiler and pump, the pump man may suggest that in a somewhat similar case he had seen the leak stopped by inserting one or more 4-in. pipes into the crevices, letting the dam fill up and then running grout into the pipe mixed with very short chopped old rope. The dredge runner may suggest that he had traced the source of leaks outside the dam using a weighted gum blanket attached to a cord, feeling "nibbles on his line," as a fisherman does—providing, of course, the pumps are working hard enough to develop the requisite "suction" to hold the floating ball—and you can then dump the right kind of material on the indicated spot. The blacksmith may insist that ashes deposited around the cofferdam is the best method, not only to discover leaks, but to effect close them, depositing the ashes from boxes with trigger bottoms, meantime keeping up the pumping and watching for rising particles of ashes inside the dam in order to locate the source of trouble. If the blacksmith hails from Jones & Laughlin or the Carnegie Furnaces, he may say that slag sand is even better than ashes to close leaks, and especially for closing leaks through the cofferdam structure.

It is quite natural to suppose that through the usually coarse gravel and boulder beds of the Ohio and Allegheny Rivers, where the depth to the rock is often 30 ft., there must be a strong flow of water. It is not generally known, however, that our visible rivers flow over a more or less cemented structure of sand and gravel from 2 to 3 ft. thick and practically impervious to water. The dredge men and the pile driver men are aware of the existence of this comparatively hard stratum which overlies the softer sands and gravel. So far as I know this "hard bed" extends along the river for several hundred miles below Pittsburg, although it may be absent in occasional areas. At the Ashland, Ky., water-works where they put down a row of drilled wells near the shore of the river on a line about 500 ft. long for the purpose of reaching filtered water, the impervious gravel was found to be about 8 ft. thick. Rather curiously the water rose in the pipes, after they had been inserted in the drilled holes, to a height several feet above the level of the water in the river. In this instance the deeper gravel was surcharged by hard limestone springs. Large springs have been struck in the river bed at several places within the city limits.

That the subterranean discharge beneath the river bed about Pittsburg is exceedingly small has been conclusively

*Abstract of a paper read before the Engineers' Society of Western Pennsylvania, May 23, 1905.

†U. S. Engr. Officer, Locks and Dams, Monongahela River, Pittsburg, Pa.

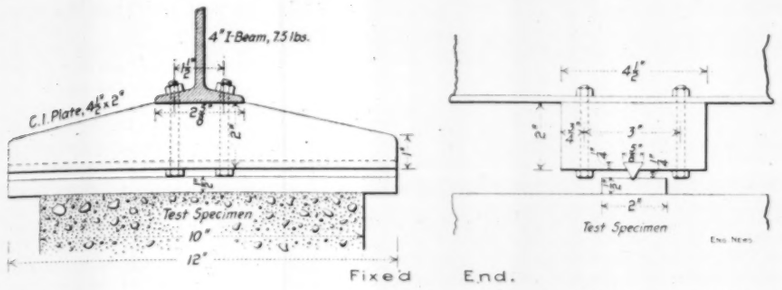


FIG. 2. FIXED AND ROCKER BEARINGS BETWEEN TEST BEAM AND LAST EQUALIZER.

demonstrated by the operations of the United States Engineers in the construction of locks and dams. The Allegheny River below Herrs Island, foot of 20th St., Pittsburg, was, during the construction of the dam at that point, penned off with dams from shore to shore in three sections, one section at a time as a matter of course. After each of these cofferdams was settled and made reasonably water tight with inner and outer embankments, a 12-in. pump would keep any one of them dry. Excavations were made in them to the depth of about 15 ft. below the bed of the river, the excavation in no case adding

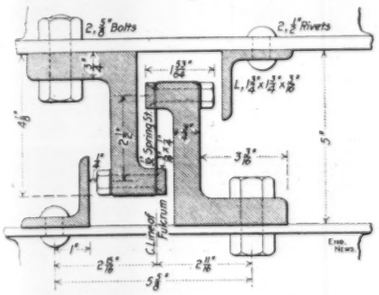


Fig. 3. Membrane Suspension at Joints of Equalizer System.

materially to the volume of water which had to be pumped. The sections unwatered covered from two to three acres each. It is thought by Mr. J. W. Arras, the engineer in charge, that fully two-thirds of the water pumped in each section came either through the cofferdams or directly beneath their bottoms where they rested on the hard bed of the river. In other words, a 12-in. pump would take care of the ground flow of the Allegheny River.

At Lock No. 2 on the Monongahela River where we have a similar dam enclosing 3.4 acres, and in which we have excavated 7 to 8 ft. below the original river bed, a 10-in. pump keeps the water out easily. There is not the slightest doubt that at No. 2, but for filtration through the dam proper above the level of the river bed, that a 2-in. pump would take care of the ground flow or discharge beneath the river bed for one-quarter of the river's width. In our work, on 250 ft. of the foundation for the dam, there was no appearance whatever of water rising from the bottom, a small siphon being able to pump out a trench 50 ft. wide, 250 ft. long, excavated 6 to 8 ft. below the bed of the river. Excavation can be done in such large dams about as easily and as cheaply as on dry land—locomotives and dump cars running on dry road beds within them. Precisely the same experience was had at Davis Island on the Ohio, and at other dams below it where extensive areas were pumped out, and excavations made beneath the river bed. Occasionally small springs are struck in the dams and I know of many places where the geological synclines cross the river, where numerous large springs occur in the river bed, but so far the government engineers have been lucky enough to miss such places and can always miss them if they look for the signs of their presence.

With this explanation you will cease to wonder at the economical and simple construction of the cofferdams used hereabouts on river work where large areas must be unwatered. These dams cost from $8 to $10 per running foot and are usually 10 to 12 ft. wide, 14 to 16 ft. high, covered on top. The bottom wales are 10 ins. square, the middle one 8 ins., and the top wale about 6 ins. square. Two-inch hemlock sheeting is used, driven with mauls usually not more than a few inches into the hard river bed.

So far as the cofferdam at No. 2 on the Monongahela, opposite the Carnegie Steel Works, is concerned, the river gravel used for filling and banking it inside and out, would of itself have made an impervious dam, but the timber form in the heart of the bank is necessary because of the floods which would otherwise wash the bank away.

At No. 2, there is an admixture of clay with the sand and gravel, as brought up by the dredges, of about 1 clay to 2 sand, and 6 gravel. Now if we substitute cement for the clay we have our average concrete mixture, and therefore the aggregate of our embankments ought to be as water tight as concrete, and so, no doubt, it would be if it had been systematically deposited so as to allow none of the clay to wash out. Clay, be it remembered, is more finely ground than cement.

This brings up the subject of materials for filling cofferdams. There is such a case as having too much of a good thing. The good thing is clay, but it should not be in excess. Clean sand alone is preferable to pure clay alone for filling cofferdams. The trouble to be avoided is having the clay so strong that the smallest leak through it may rapidly develop into a self-arched tunnel.

When I recall some railroad pier cofferdams that I put in years ago, I would, if I had the jobs to do over again, advise myself as follows: Before shoveling into the dams any sand mixed with clay, endeavor first to sink a course of water logged chopped oat straw, by dropping ashes over it, or forcing the straw down with a wire net and filling the straw with ashes, or slag sand, to the depth of several inches. The first and generally the only thing needed to make a tight dam is to put some material in its bottom, which when a fluid pressure is exerted upon it will tend to choke up the outlet.

On bridge work it is seldom that it would be practicable to place the dam at distances of 50 to 100 ft. from point of maximum depth of excavation, as we like to do on the river. The streams are too narrow and besides they may not have the impervious "bone beds" as the dredge men call the hard stratum of our river beds. Hence it comes that the dams are close to the masonry line. In lieu of inside embankments, however, the bridge engineer has better and safer methods of bracing his work to resist heavy pressures than we have on our dams. To secure a good job the sheet piling must be very carefully driven. It is better to lose a half day's time pulling out a stick which has wandered, and putting in its place a stronger one, probably with an iron shoe on it. Whenever possible the piles should pass between two lines of waling or guides, the guides as far apart vertically as practicable and every pile, while being driven, kept tightly clamped with a chain to those already in place. Care and patience count more than anything else with sheet pile driving.

I will remark in conclusion that steam pile drivers are much to be preferred to drop hammers. A pile which goes down with a uniform speed is less apt to jam, buckle, or split than when driven with heavier blows with marked intervals of time between blows. I base this

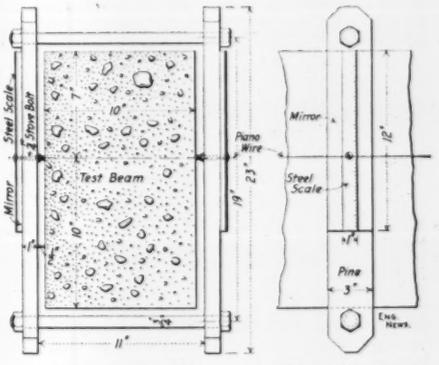


Fig. 4. Mirror-and-Scale Yoke for Measuring Deflections of Test Piece.

statement on what I have observed in putting down steel testing rods by hand. Thus through soft gravel mixed with quicksand a man may be able to push a rod down 5 or 6 ft., and if quick enough with the same expenditure of power pull the rod up again. If, however, the rod is left to stand, even no longer than 15 seconds, the sand packs against it so that two men may scarcely be able to pull it up. A pile therefore left standing in some kind of sand for a few moments may become so packed as to refuse to be driven further, or at least require much greater force to be started again. This packing of sand about piles impresses me with the belief that we may often underestimate their stability and the loads they might safely carry.

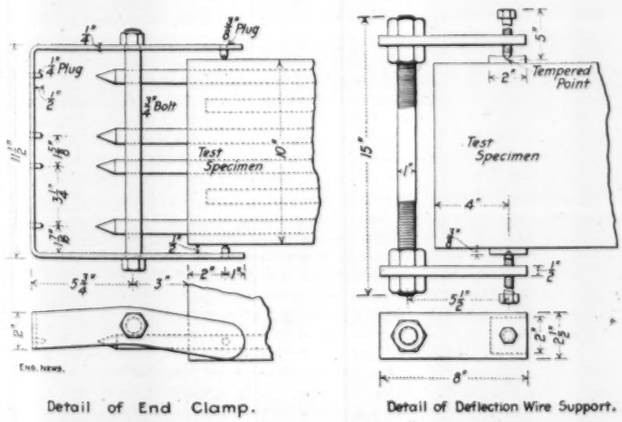


FIG. 5. DETAILS OF DEFLECTION WIRE SUPPORT AND END CLAMP FOR MEASURING SLIP OF REINFORCING BARS.

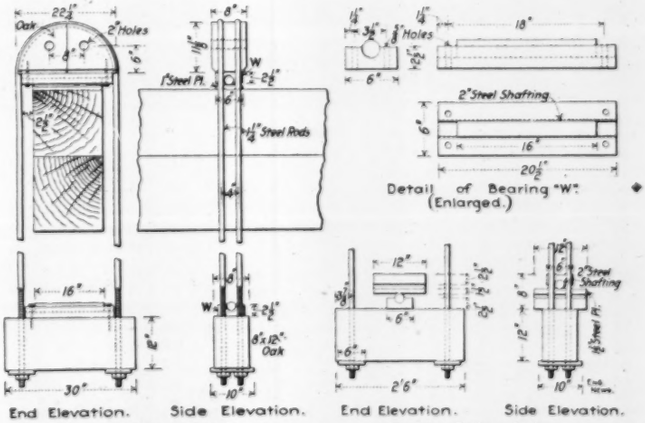


FIG. 6. ROLLER-JOINT STIRRUPS FOR SUPPORTING ENDS OF TEST BEAM.

In conclusion I will remark that in one treatise on cofferdams that fell under my observation it was said that the engineer should regard them as retaining walls and calculate for their overturning moment accordingly. I think this remark should have been qualified to read that a cofferdam is like a retaining wall, only one is begun at the top and the other at the bottom. The engineer must stand on his head, so to speak, to fully comprehend the similarity of the two kinds of structures. If even some of the best designed retaining walls commenced at their bottom on dry ground, or rock, upset occasionally or slide on their bases, there ought to be more excuse for the engineer who begins to build his up side down retaining wall, or cofferdam, at the top, while to a great extent ignorant of what is going to be found as his work proceeds, if said work takes a notion to burst in while it is being pumped.

THE EFFECT OF CLAY AND LOAM ON THE STRENGTH OF PORTLAND CEMENT MORTARS.*

By W. C. Hoad.†

Frequently, on work involving concrete or masonry construction, dirty sand—that is, sand with a small admixture of clay or fine silt—is the only kind practically available, and the question as to whether such sand may be properly used as it is, or whether it must be washed before being used, is often a very troublesome one to the inspector in charge of the work, as well as to the engineer who draws up the specifications. Under circumstances of this kind, there naturally arises the question, How much does this dirt in the sand detract from the strength of the resulting mortar? and until recently reliable data on this point were almost entirely lacking. There has been, it is true, a feeling among the users of mortar that perfectly clean sand was the best for this purpose, and that any adulterant, like dust or dirt, was injurious. Indeed, the idea has become so firmly rooted that the specification that "the sand used on this work shall be clean, sharp and coarse" has almost hardened into a formula. But, as the thorough washing of dirty sand is an expensive operation, the cost of structures where this specification is enforced is obviously increased, sometimes by a rather large percentage of the total cost.

An investigation into the subject was undertaken last winter, as a graduation thesis in the department of civil engineering at the University of Kansas, by Messrs. Ben. C. Hoefer and Earle Nelson. This investigation consisted for the most part of a series of tests of the tensile strength of mortar made of Portland cement and sand containing various proportions, from 2 to 20%, of clay or loam. The cement experimented with was the Iola brand of Portland cement, a cement much used in engineering construction in this part of the country. The sand used was the standard Ottawa sand adopted for cement testing by the American Society of Civil Engineers. The clay was the ordinary reddish clay commonly found on the hillsides west of the University campus and the loam was from the top soil of a corn-field.

In making the mortar briquettes for testing, the proportion of one part of cement to three parts of sand was used. When clay or loam was added, it was taken as a certain percentage of the sand, and thus the ratio of cement to sand, or of cement to clay or loam and sand, was maintained constant. All proportions were by weight.

At the end of three, seven, twenty-eight and ninety days sets of briquettes of the various proportions of clay and loam were broken in a late model Falkenau-Sinclair automatic cement-testing machine. The results for the clay mixtures are shown graphically in Fig. 1, while those for the loam mixtures are shown in Fig. 2. Each set consisted of six briquettes, the mean of the three strongest briquettes in each set being taken. Each point in the diagrams represents one set.

TABLE I.—Showing Absorption of Mortars With Clay and Loam Added.

Per cent. of clay or loam.	Per cent. of water absorbed in twenty-four hours. Clay.	Loam.
0	5.5	4.5
2	4.0	4.3
4	3.9	4.4
6	4.8	4.6
10	4.9	7.4
15	6.1	7.0

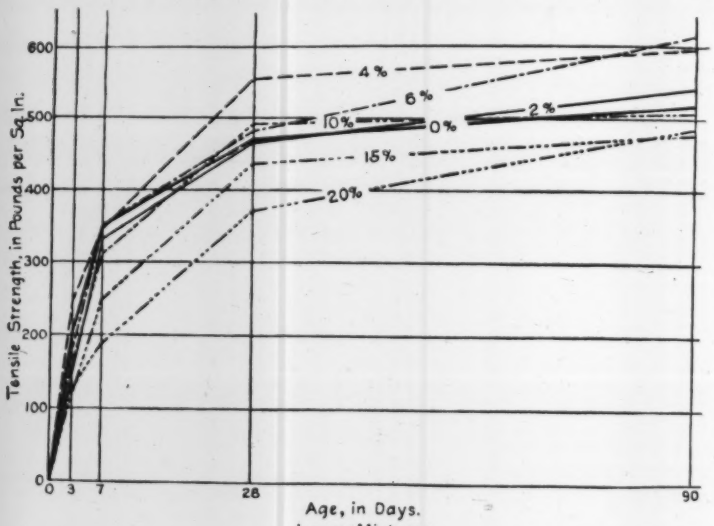
These tests indicate clearly that the presence of clay or loam in the small amounts usually found in dirty sand (say from 2 to 10%) does not detract from the strength of the mortar, but rather adds to it. There seems to be no important difference between the effect of clay and that of loam. It is also apparent from the diagrams that short-time tests of this nature are not of much value.

Table I. gives the amounts of water absorbed by the briquettes in 24 hours, stated as percentages of the dry weights. These results, while not conclusive, point to the inference that one-to-three mortars made with sand containing up to 5 or 6% of clay or loam are at least as dense as those made with clean sand. With leaner mortars—that is, those with a smaller proportion of cement—the greater density of the dirty mortars would doubtless be more apparent. It will be noted that the effect of clay in increasing the density of mortar is noticeably greater than that of loam. This is doubtless due to the exceedingly small size of the clay grains, enabling them to fill the interstitial spaces of the cement.

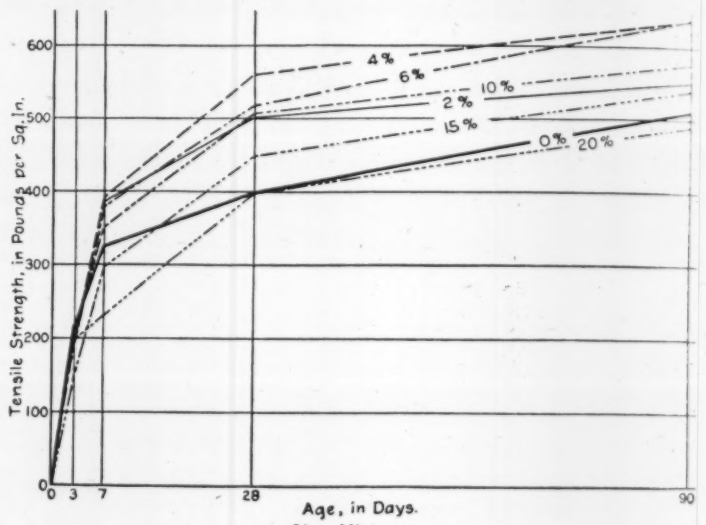
In view of the data above presented or referred to, it would seem to be a waste of money to require small amounts of earthy material to be washed out of sand that is to be used for Portland cement mortar, whether that mortar is to be used in ordinary masonry or in concrete. An exception to this conclusion might be taken if the mortar were to be placed under water, or if a quick-setting mortar were desired, but experimental evidence on these points is still lacking. For the general uses of cement mortar, the conclusions stated above would hold good.

*A paper read before the Academy, at Topeka, Dec. 30, 1904. See also Engineering News, Vol. 50, p. 443, and Vol. 53, pp. 127, 231.

†University of Kansas, Lawrence, Kan.



Loam Mixtures.



Clay Mixtures.

FIGS. 1 AND 2. DIAGRAM SHOWING TENSILE STRENGTH OF CLEAN SAND MORTAR, COMPARED WITH MORTARS HAVING DIFFERENT PERCENTAGES OF ADDED CLAY AND LOAM.

ELECTRIC HEADLIGHTS FOR LOCOMOTIVES.

The use of electric headlights for locomotives has increased very extensively during the past few years, and practical experience has proved their advantages. They not only enable enginemen to see obstructions, misplaced switches, etc., at such a distance as to enable trains to be stopped before reaching them, but they are effective in preventing collisions, while the brilliant beam of light is a warning to persons on the track or at grade crossings. The Pyle electric headlight is in use on about 5,000 engines, over 90% of which are in this country, Canada and Mexico, and this was described and illustrated in our issue of June 16, 1898. The Edwards headlight, described below, is of more recent date, and its generating plant is said by some of the railways which have used it to be more economical in steam consumption. It also has the peculiarity of being able to throw a vertical beam of light, which can be seen at great distances and is claimed to be a safeguard against collisions, though some railways (while approving of the headlight) do not regard this feature as being very important. About 250 of these headlights are in use. In both the above systems, the dynamo for generating current is driven by a steam turbine, and the De Laval Steam Turbine Co. has recently designed a turbine-dynamo equipment for locomotive headlights, but this has so far only been in limited experimental use.

In the Edwards system, a 14-in. turbine-wheel is mounted on a shaft coupled directly to the dynamo, and this generating equipment may be mounted on the same base as the lantern, as in Fig. 1, or it may be mounted on a separate base plate and located at any convenient place on the engine. The turbine, Fig 2, consists of a single cylindrical chamber, within which revolves a steel wheel fitted with die-forged buckets on the rim. The exhaust is almost opposite the steam port, and the engine is said to work at a very fair steam economy. The normal speed of engine and dynamo is about 2,000 revolutions per minute. The shaft is carried in ball bearings, so that there is very little frictional resistance, and the bearings are lubricated by loose rings hanging in oil chambers. A governor keeps the speed within a range of 3% from the normal, under any conditions of pressure or load. A ¾-in. steam pipe is led from the cab to the turbine, while a 1¼-in. exhaust pipe is led into the the smokebox. The dynamo is specially designed for the work. The field is differentially wound and the electric circuits are so arranged that a burned-out armature is said to be an impossibility. Should a short circuit occur at any point, the current is killed, and though the armature may still be rotated by the engine, no current will be generated until the short circuit is removed. The current densities throughout the machine are very low, so that a minimum heat effect is produced, while the electrical balance is so well adjusted that no sparking occurs at the brushes. Carbon brushes of low resistance and high durability are used. The shaft has long and large journal bearings, lubricated in the same way as those of the turbine shaft.

The lamp is designed to give a steady and reliable light without flickering, taking a current of 30 to 33 amperes and 30 to 33 volts. It has a carbon positive and copper negative, the former being fitted with an adjustable guide for maintaining the focus by means of special mechanism. One of the striking features of the headlight is the use of a deflector in front of the upper part of the lantern, inclined at an angle of about 45° with the lens. This intercepts about 40% of the light and projects it as a vertical beam, which is conspicuous for a distance of several miles. The horizontal beam lights up the track, on a

tangent, for a distance of ¾ to 1 mile ahead of the engine. The light is found not to obscure the switch or signal lamps or to render their colors less distinct. It is sometimes objected that on double track the enginemen of opposing trains would be dazzled by the electric light, and to meet this objection the lamp is provided with a semi-

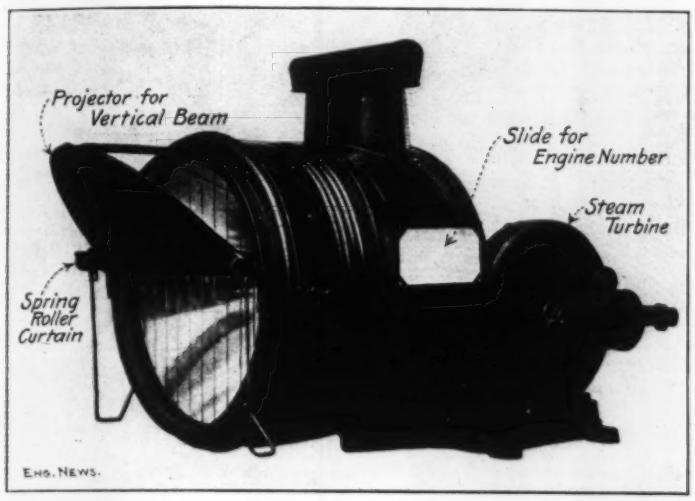


FIG. 1. THE EDWARDS ELECTRIC HEADLIGHT, WITH TURBINE ATTACHED.
Edwards Railway Electric Light Co., Cincinnati, O.

transparent curtain on a spring roller in front of the glass, as shown in Fig. 1. This is operated from the cab, and the engineman draws it down when approaching a train on the opposite track, reducing the intensity of the light, as though obscured by ground glass. As soon as the trains have passed the shade is released.

This electric headlight equipment is the invention of Mr. E. A. Edwards, and is manufactured by the Edwards Railway Electric Light Co., Monadnock Block, Chicago, with factory at Laporte, Ind. As already noted, it is now in use on about 250 engines. Articles on the efficiency and economy of electric and other headlights will be found in our issues of July 2, 1903, and May 19, 1904.

INVESTIGATION OF STRESSES IN HIGH MASONRY DAMS OF SHORT SPANS.*

By Geo. Y. Wisner,† M. Am. Soc. C. E., and Edgar T. Wheeler,‡ M. Am. Soc. C. E.

GENERAL DISCUSSION BY MR. WISNER.

In a paper presented at the Conference of Engineers of the Reclamation Service at Ogden in September, 1903, the writer called attention to the necessity of having a thorough investigation made to determine the stresses which will be developed in high masonry dams by changes of temperature and the combination of masonry and steel reinforcement requisite for absolute safety of such structures with minimum cost of construction.

In compliance with a recommendation made by the Board of Consulting Engineers, October, 1904, the writer was authorized to employ Mr. Edgar T. Wheeler, C. E. of Los Angeles, Cal., to make such an investigation under his direction. Mr. Wheeler took up this work under the supervision of the writer in January, 1905, and his report submitted herewith indicates clearly the nature and location of the stresses which must be provided for in short masonry dams of the arch type, and also that economical construction consistent with safety requires independent design for every structure having different dimensions, or if constructed of materials having different strength, weight and coefficient of expansion and elasticity.

Any attempt to generalize designs for such structures is almost certain to be of no value except for conditions identical with those on which these designs were based, and any theoretical determination of stresses and pressures based upon assumed conditions can not be otherwise than worthless until modified so as to conform to the conditions and materials in the structure where applied.

The formulae given in most reference books for determining the stresses in dams of different dimensions are generally for those requiring a gravity section, or for long straight dams, and it is generally assumed that if safe for long dams, such gravity sections will have greater stability for short straight dams and for arch dams, and, therefore, such sections have been adopted in cases where equally stable structures could have been erected at much less cost if actual conditions had been given proper consideration.

It is often stated that the assumptions made are on the side of safety. This, however, is by no means satisfactory, for while an engineer may adopt a design for a structure which has been proven absolutely safe and satisfactory under other conditions, he may by such a course expend thousands of dollars unnecessarily in making the structure massive without increasing its stability.

The first duty of the engineer in making designs is to demonstrate beyond all question that the proposed structure will be safe under all the internal and external forces to which it can be subjected, and that its parts are so proportioned that no unnecessary expense will be incurred by the construction of portions which will not add to its stability.

The assumption often made that a dam is an absolutely rigid structure and that the foundation on which it rests is perfectly elastic is by no means in accord with the materials used in construction, or with those in any foundation suitable for supporting such a structure. Under this assumption the pressures on the foundation will always be the same for any given stage of water in the reservoir, whereas in a dam of the arch type internal stresses arising from a change of temperature of the structure may cause a wider range of pressure on the foundation than that arising from any possible change of water pressure on the face of the dam (see Fig. 1).

If the structure, or the foundation on which it rests, is perfectly rigid the diagram of pressure on the base will be entirely different from what is the case where the material in each has the elasticity and compressibility of ordinary masonry. If a, b, c, in Fig. 1, represents a section of a symmetrical rigid structure resting upon a perfectly homogeneous elastic foundation free to move vertically under pressure, the diagram of pressure on the foundation will be a rectangle. If the foundation material is not homogeneous the points of greatest pressure will be those offering the greatest resistance to compression. If these points are near the center of the base the diagram will be convex and if near the ends it will be concave showing a greater pressure at a and b than at the center, d.

In case the material of the foundation is compressible with no shearing strength along the faces of the dam at a and c, the structure will settle until the resistance of the foundation to compression is equal to the weight of the structure; but if the material of the foundation is continuous outside of the base and elastic, the pressure from the total load will be resisted by the elastic forces in the material of the foundation and the shearing resistance developed at the points a and c. A structure having a pressure of 100 lbs. per sq. in. on its base, and a modulus of elasticity for foundation of 3,000,000, will compress the foundation 0.1-in in depth of 250 ft. if free to act without shearing resistance at a and c. This compression will be resisted, however, by the shearing strength of the foundation at a and c, and therefore, if as assumed, the structure is rigid the greatest upward pressures under the base will be at a and c and the minimum at the center, d.

The assumption that such structures will be rigid is far from being correct and may lead to dangerous results. The material of which dams are usually constructed will compress practically the same as that of the foundation and in a dam of symmetrical section, like that shown, Fig. 1,

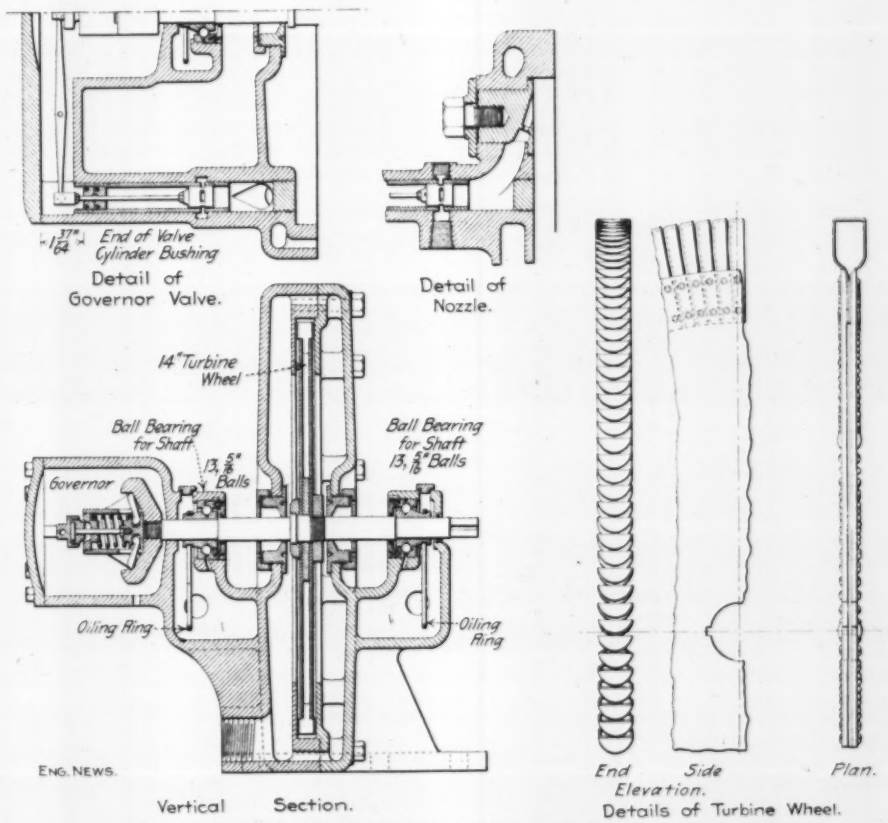


FIG. 2. STEAM TURBINE FOR EDWARDS ELECTRIC HEADLIGHT.

*A report made to Mr. F. H. Newell, M. Am. Soc. C. E., Chief Engineer of the U. S. Reclamation Service, May, 1905.

†Consulting Engineer, U. S. Reclamation Service, 34 W. Congress St., Detroit, Mich.

‡Consulting Engineer, Los Angeles, Cal.

the maximum pressure will be at d, and the minimum at the faces a and c, or the reverse of the result obtained by considering the structure absolutely rigid and resting upon an elastic foundation.

In the case of a dam of ordinary gravity cross section, with the resultant line of pressure cutting the base one-third the width of the dam from either face, the load will be supported in part

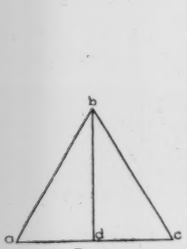


Fig. 1.

Fig. 2.

by the elastic forces in the foundation and partly by the shearing forces at the heel and toe of the dam developed by the foundation strata outside of the base carrying a part of the load. It is therefore evident that the static pressure at the toe of a dam of ordinary theoretical gravity section is much greater than shown by the formulae in reference books, in which formulae the effect of shearing strength in the foundation is neglected, making it important that as much of the load should be transmitted to the side walls as possible.

A foundation suitable for supporting a high masonry dam must be continuous for some distance up and down stream from the structure. The weight of the dam will not be carried entirely by the material of the foundation immediately under the base, but will be distributed through the strata above and below the dam by the bending action produced by the slight compression of the rock directly under and adjacent to the structure.

In the construction of a masonry or concrete dam a foundation course 40 to 50 ft. in depth is usually put in place for the entire length and width of the structure, which in a short dam, when

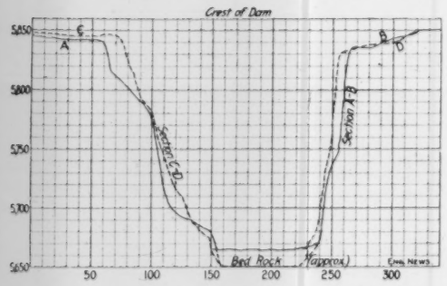


Fig. 4. Cross-Section of Canyon at Pathfinder Dam-Site.

thoroughly set, becomes a rigid beam, slightly compressed under heavy loads. The mid-stream portion of the dam can not compress the amount that it would if free to move, on account of the shearing stresses developed at the side walls by the beam action of the lower portion of the dam. Any unequal distribution of the load on the mid-stream section will produce both compression and torsion in the foundation course.

The assumption that the diagram of pressures under the base of a dam of theoretical cross section is a triangle with zero pressure at the toe when the reservoir is empty requires that the section under consideration is free to move. In short dams this is not the case and any such movement will create bending moments, compression and torsion in the lower part of the structure impossible to obtain with the loads to which subjected. The lower portion of a short masonry dam is little more than a massive wedge between the walls of the gorge which the dam closes and if of sufficient thickness to safely withstand the shearing stresses to which subjected it will be just as stable as though made with a massive gravity section.

In designing such structures, careful consideration must be given to the effect of changes of temperature of the structure above or below that when built, which may develop greater stresses in the masonry than those due to the water pressure on the face of the dam. The lower portion of the dam, which is massive, protected from the sun and saturated with water from the face of the dam, will have but a small range of temperature for the entire year, while the upper portion, which is thin and exposed, will have a wide range. The results of these conditions are that in the summer season the upper portion of such a dam is under heavy compressive strains, and in the winter it is subject to equally strong tensile strains.

Good concrete will contract or expand about 1-200,000 for each degree change of temperature, and the co-efficient of expansion of good building stone varies from 1-175,000 to 1-500,000 for each degree change.

A masonry structure having a co-efficient of expansion of .000,005, and a modulus of 3,000,000, will develop tensile stresses of 15 lbs. per sq. in. for each degree fall in temperature below the normal, or over 21 tons per sq. ft., for a temperature 20 degrees lower than that at the time the masonry was constructed, a condition that clearly indicates that the upper portions of such structures should be constructed when the temperature is below the normal for the location for which designed.

These forces have a tendency to give the crown of the arch at the crest of the dam a slight movement up and down stream, and since the lower portion of the structure is not subject to these changes of temperature, the movement produces bending moments in the vertical sections of the dam, which if not reinforced with steel may result in horizontal cracks in the down stream face and vertical cracks through the upper part of the structure.

The general use of the same design of section for short dams of the arch type as for long dams requiring a gravity section is open to criticism, for the reason that the up-stream pressures due to change of temperature of the upper part of the dam may exceed those down stream, due to the water pressure on the face of the dam. If the local conditions at the site are such that the dam can be designed advantageously with a radius of 200 ft. or less, the batter of the up-stream face should be at least one-half that of the down-stream face, and in dams of very short spans a similar batter on each face gives a good distribution of stresses, when properly reinforced with steel.

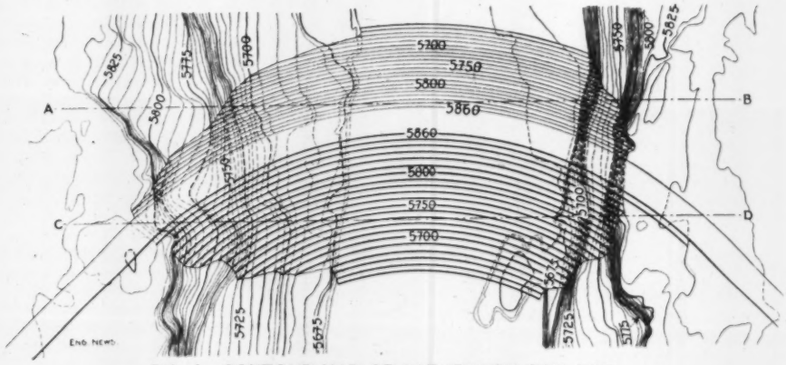


FIG. 3. CONTOUR MAP OF PATHFINDER DAM-SITE.

The accompanying diagrams showing the distribution of pressures to the side walls and to the base of the structure indicate clearly that the safest and most economical construction for short dams, where the rock in the walls is continuous and solid, is a structure having sufficient thickness to be absolutely safe from shear under all forces to which it can be subjected, with the upper portion so reinforced with steel as to be free from danger of cracks from bending moments and from temperature strains.

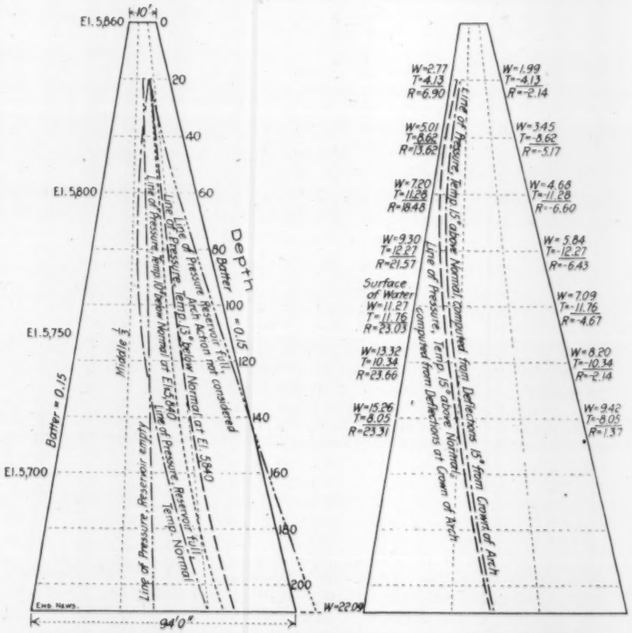


FIG. 5. PROFILES OF PATHFINDER DAM, SHOWING LOCATION OF LINES OF PRESSURE UNDER DIFFERENT CONDITIONS; ALSO SHOWING PRESSURE DUE TO CHANGE OF TEMPERATURE.

W = Pressure per sq. ft. due to weight of masonry.
T = Pressure per sq. ft. due to rise of Temperature of 15° at top of dam; reservoir filled to elevation 5,760.
R = Resultant pressure per sq. ft.
Pressures given correspond to line of pressure computed from deflections at points 15° from crown of arch.
(See also note to Figs. 6 to 9.—Ed.)

Mr. Wheeler's investigations were made with especial reference to the conditions at the Pathfinder dam site, and to be made applicable to other locations the batter used for the faces would have to be changed to give the proper thickness to resist shearing stress, and the radius

of curvature changed to conform to the length of span required.

It was found that, at the Pathfinder site, a batter of 0.35 of the height, with a thickness of 10 ft. at the top of the dam, gave a width of structure such that the end shear at any horizontal section was less than one-tenth the safe compression load for the masonry to be used; or, in other words, the dam would be safe under arch action alone, without taking into consideration the weight of the structure.

A radius of 160 ft. for the up-stream face of the dam was used in the preliminary investigation of the distribution of pressures arising from water pressure on the face of the dam and from changes of temperature of the structures itself. The methods by which these lines of pressure were determined are clearly explained in Mr. Wheeler's report and shown on the accompanying diagrams.

The preliminary investigation of a dam 10 ft. thick at the top, with a down-stream batter of .30 and up-stream batter of .05, demonstrated that a dam of such a cross-section would be safe, but that a better distribution of the lines of pressure would be obtained by making the batter of the faces nearly equal.

At a conference of the Board of Consulting Engineers (of the U. S. Reclamation Service.—Ed.) in March it was decided to recommend a design for the Pathfinder dam, having a top width of 10 ft., a down-stream batter of .25, an up-stream batter of .15, and a radius of 150 ft. for the center of the top of the dam.

With this plan and profile of the dam, the distribution of pressure was determined for the reservoir full and empty for normal temperatures and for temperatures 15 degrees above and 10 degrees below the normal, the results of which are shown on the accompanying diagrams.

The first diagram in Fig. 5 shows that for normal temperature and with the reservoir full, the resultant line of pressure for that portion of load supported by the foundation falls within the middle third, while considered as a gravity section with only its own weight to resist the water pressure, the resultant falls outside of the base. Referring, however, to the axial pressures given

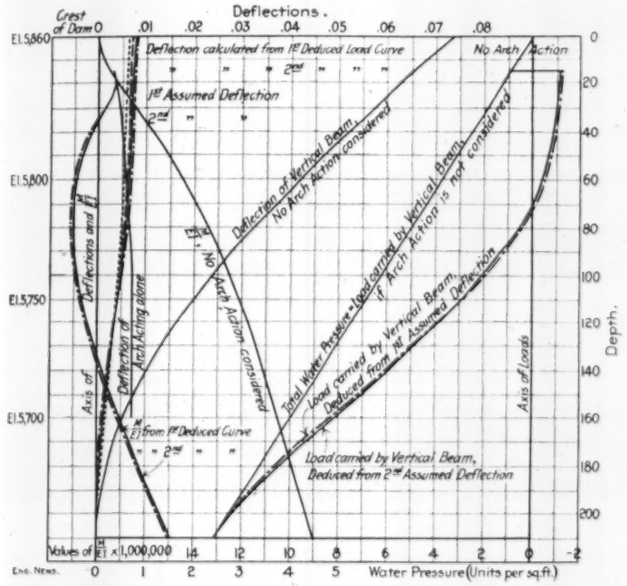

Deflections.
Crest of Dam
El. 5860
El. 5800
El. 5750
El. 5700
Deflection calculated from 1st Deduced Load Curve
No Arch Action
1st Assumed Deflection
Deflection of Vertical Beam, No Arch Action considered
Deflections and M/EI
Deflection of Arch Acting alone
Axis of M/EI
M/EI from 1st Deduced Curve
No Arch Action considered
Total Water Pressure = Load carried by Vertical Beam, if Arch Action is not considered
Load carried by Vertical Beam, Deduced from 1st Assumed Deflection
Axis of Loads
Load carried by Vertical Beam, Deduced from 2nd Assumed Deflection
Depth.
Values of M/EI x 1,000,000
Water Pressure (Units per sq. ft.)
ENG. NEWS.

FIG. 6. RESERVOIR FULL; CHANGE OF TEMPERATURE NOT CONSIDERED.

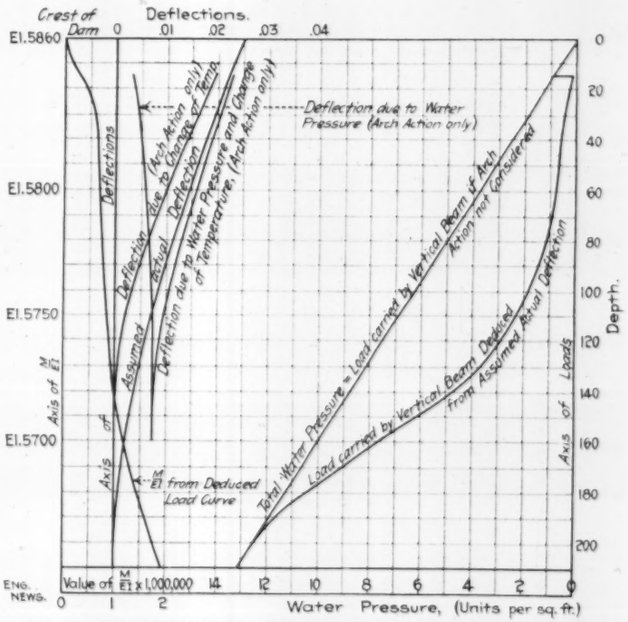

Deflections.
Crest of Dam
El. 5860
El. 5800
El. 5750
El. 5700
Deflection due to Water Pressure (Arch Action only)
Deflections (Arch Action only) due to Change of Temp.
Actual Deflection due to Water Pressure and Change of Temperature, (Arch Action only)
Assumed
Axis of M/EI
M/EI from Deduced Load Curve
Total Water Pressure = Load carried by Vertical Beam if Arch Action not considered
Load carried by Vertical Beam Deduced from Assumed Actual Deflection
Axis of Loads
Depth.
Value of M/EI x 1,000,000
Water Pressure, (Units per sq. ft.)
ENG. NEWS.

FIG. 7. RESERVOIR FULL; TEMPERATURE OF DAM 10° BELOW NORMAL AT POINT 20 FT. BELOW CREST.

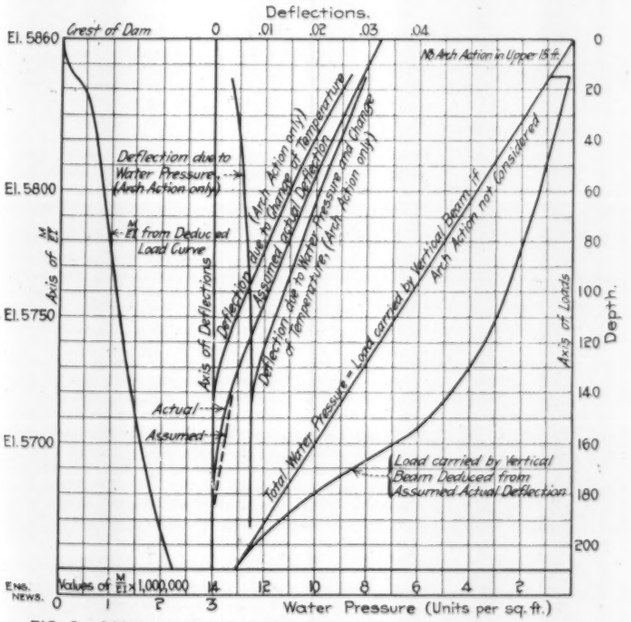

Deflections.
Crest of Dam
El. 5860
El. 5800
El. 5750
El. 5700
No Arch Action in Upper 15 ft.
Deflection due to Water Pressure, (Arch Action only)
M/EI from Deduced Load Curve
Axis of M/EI
Axis of Deflections
Deflections (Arch Action only) due to Change of Temperature
Assumed Actual Deflection
Deflection due to Water Pressure and Change of Temperature (Arch Action only)
Actual
Assumed
Total Water Pressure = Load carried by Vertical Beam if Arch Action not Considered
Load carried by Vertical Beam Deduced from Assumed Actual Deflection
Axis of Loads
Depth.
Values of M/EI x 1,000,000
Water Pressure (Units per sq. ft.)
ENG. NEWS.

FIG. 8. SAME AS FIG. 7, EXCEPT TEMPERATURE OF DAM 13° BELOW NORMAL.

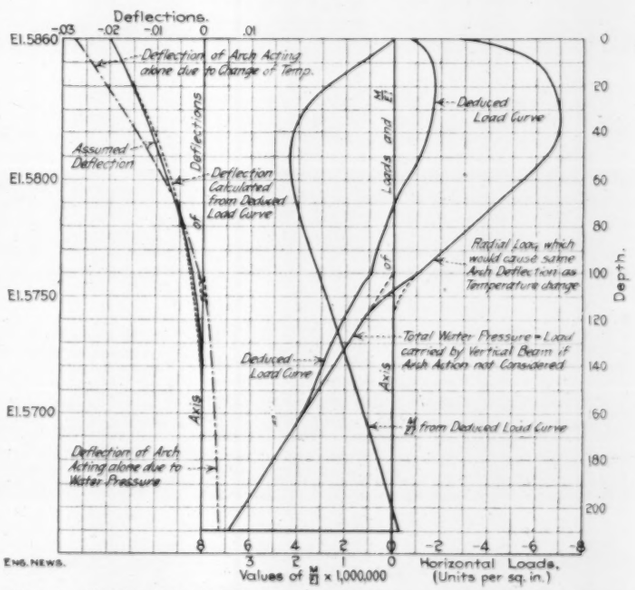

Deflections.
El. 5860
El. 5800
El. 5750
El. 5700
Deflection of Arch Acting alone due to Change of Temp.
Assumed Deflection
Deflections
Deflection Calculated from Deduced Load Curve
Deduced Load Curve
Axis of M/EI
Loads and
Radial Load, which would cause same Arch Deflection as Temperature change
Total Water Pressure = Load carried by Vertical Beam if Arch Action not Considered
Deduced Load Curve
Axis
M/EI from Deduced Load Curve
Deflection of Arch Acting alone due to Water Pressure
Depth.
Values of M/EI x 1,000,000
Horizontal Loads, (Units per sq. in.)
ENG. NEWS.

FIG. 9. RESERVOIR FILLED TO ELEVATION 100 FT. BELOW CREST OF DAM; TEMPERATURE AT CREST OF DAM 15° BELOW NORMAL.

FIGS. 6 TO 9.—DIAGRAMS USED IN STUDYING PATHFINDER PROJECT, WITH GRAVITY SECTION CONSIDERED AS A VERTICAL BEAM; SHOWING DISTRIBUTION OF LOAD BETWEEN ARCH AND GRAVITY SECTION, DEFLECTIONS AND MOMENTS.

EXPLANATION OF DIAGRAMS.

Unit of weight = 1,000 lbs.
Unit of length = 1 ft.
Weight of concrete = 153 lbs. per cu. ft.
E of concrete = 432,000 equivalent to 3,000,000 in pound and inch units.

Coefficient of expansion of concrete = 0.0000066 for 1° F.
Weight of water = 62.5 lbs. per cu. ft.
Pressure of water considered as acting horizontally on face of dam.
In computing location of lines of pressure the horizontal sections of dam are considered as being bounded on the sides by parallel instead of radial planes.
In computing deflections of the vertical beams, each vertical beam is considered as bounded on its sides by radial planes.

in Mr. Wheeler's report, and to the small deflection of the dam from arch support alone, shown in Fig. 6, it is evident that its stability, as a horizontal arch beam, is ample to withstand the water pressure in the reservoir for all conditions.

With the temperature 10 degrees below the normal, so that the tensile stress in the upper portion of the dam is acting with the water pressure of the full reservoir, the resultant line of pressure falls outside of the middle third for height of 30 ft. above the base. This, however, is in that part of the dam which acts simply as a wedge between the walls, and no doubt distributes a larger amount of the pressure to the side walls than shown by the analysis.

The second diagram, in Fig. 5, shows the line of pressure for the water 100 ft. deep in the reservoir, and the temperature 15 degrees above the normal at the top of the dam, decreasing to a normal temperature 120 ft. below the top of the dam. This is about as bad a condition as can be assumed for producing bending moments in the vertical section, and is particularly interesting from the fact that it indicates the location and amount of steel necessary to prevent horizontal cracks in the down-stream face of the dam. The amount of horizontal tensile stress which will be developed in the upper portion of the dam by the temperature of the structure falling below that at which constructed is somewhat difficult to determine. If the horizontal arch sections of the dam are free to move in all directions no stress will be developed by a change of temperature, but if the ends of the arches are fixed firmly at the side walls and the crown prevented from moving the tensile stress developed by a fall of temperature will equal the coefficient of expansion, × modulus of elasticity, × the number of degrees change of temperature, or for ordinary masonry about one ton per square foot for each degree of change. The actual stress developed will be this maximum stress corrected for the movement of the crest of the dam due to bending moments in the vertical sections, and for the effect of the bending moments in the horizontal arches. The deflection of the crest of the dam by stresses due to change of temperature can be determined by an analysis similar to that shown by Figs. 6 to 9, and the resultant horizontal stress in the arches determined with sufficient accuracy for estimating the amount of steel necessary to prevent vertical cracks in the masonry.

The probable range of temperature of the masonry must be determined for each structure, and the amount of steel necessary to prevent cracks estimated on the basis of the stresses which will be developed.

Less steel will be required to prevent horizontal cracks on the down-stream face where a uniform batter is used than in dams where the amount of batter increases from the top downward. The specifications for the Salt River, Pathfinder, and Shoshone dams provide that the contractors shall place such steel in these structures as the United States may furnish for the purpose of reinforcing the upper portions where tensile stresses are likely to develop from the range of temperature at the respective locations.

The mean change of temperature of a mass of masonry which will take place for any given atmospheric change is not well determined, and all possible data bearing upon this question should be obtained during the construction of the lower portions of these dams for the purpose of determining the amount of steel which will be required in the upper portions. Thermophones placed in the adjacent side walls and masonry when constructed and read at frequent intervals, will furnish valuable information in connection with the structure itself, and be of great value in making design of other dams.

MR. WHEELER'S ANALYSIS OF STRESSES IN THE PROPOSED PATHFINDER DAM.

Sir: I submit herewith the results of the analysis of stresses in the proposed Pathfinder Dam of the North Platte Project, Wyoming, which I have been making under your direction.

The profile of dam considered in this investigation, is a trapezodial profile, with

Width of crest	= 10 ft.
Batter of up-stream face	= 0.15
Batter of down-stream face	= 0.25
Radius of up-stream face at crest	= 135 ft.

This profile is shown by Fig. 4.

The notation used is as follows:

FOR ARCH.

D = deflection of arch due to load or change of temperature.
B = one-half the angle subtended by arch.
R = radius of up-stream face.
R' = radius of neutral axis.
L = depth of arch.

FOR GRAVITY SECTION.

P = water pressure (horizontal) per square foot.
D = horizontal deflection of section considered as a vertical cantilever.
E = modulus of elasticity of masonry = 432,000, equivalent to 3,000,000, using pound and inch units.
M = bending moment in vertical beam due to the horizontal pressure.
Unit of weight used is 1,000 lbs.
Unit of measurement 1 ft.
Weight of masonry (concrete—Ed.) assumed, 153 lbs. per cu. ft.

The following method has been used to determine the distribution of the load or water pressure between the arch and the dam considered as a vertical beam.

The dam is first considered as a succession of horizontal arches free to move relatively to each other, and the deflection of the crown of each arch due to the water pressure at its respective depth is computed by the formula developed by Mr. R. Shirreffs (see Trans. Am. Soc. C. E., Vol. LIII., p. 162).

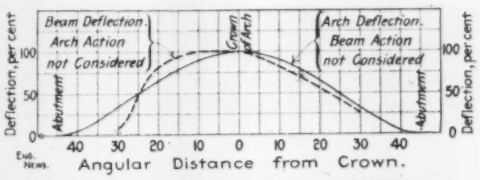


Fig. 10. Comparative Deflections at Different Points on Crest of Pathfinder Dam.

(Deflections are given in percentages of deflection at crown. The diagram does not indicate that the actual beam deflection at the crown is the same as the arch deflection.)

This formula for an arch fixed at both ends and subjected to a uniform radial load, P, is

$$D = \frac{C P R R}{E L}$$

in which

$$C = \frac{\frac{2 \sin B}{B}(1 - \cos B) + \frac{\cos 2B - 1}{2}}{\frac{3B}{\sin B} + \cos B - 4} + 1 - \cos B$$

These deflections are then platted to scale for different depths and a curve of deflections, AB, Fig. 2, is drawn.

Next the actual deflection of the dam is assumed, and the curve of assumed actual deflections, C D., is drawn.

Taking any point, a, in the dam, let

ac = the deflection due to a radial load P (arch action alone being considered), and

ab = the actual deflection

Then the actual load, P', carried by the arch $= P \frac{ab}{ac}$ (1)

And the actual load, P'', carried by the vertical beam $= P \frac{bc}{ac}$ (2)

Using equation (2) the loads P'' carried by the vertical beam can be computed, and the deflection of the vertical beam due to loads P'' can be platted. If this deflection coincides with the assumed deflection, C D, the loads P'', as computed, are correct. If it does not coincide with C D., another actual deflection must be assumed and P'' recalculated, this operation being repeated until the two lines practically coincide.

The deflection of the dam considered as a vertical beam is computed by the following method:

Values of $\frac{M}{EI}$ are computed for horizontal sections of a slice of the dam 1 ft. long at up-stream face;

M being the moment due to the water pressure,

I the moment of inertia of the horizontal section.

These values are platted, the values of $\frac{M}{EI}$ being laid off on the X axis and the corresponding depths on the Y axis; the deflection of any point of the vertical beam will be

$$D = a \Sigma \frac{M}{EI}$$

a being the distance from the point whose deflection is required to the center of gravity of the area included between the $\frac{M}{EI}$ curve, the Y axis, the X axis and the horizontal through the said point.

$\Sigma \frac{M}{EI}$ is of course represented by the above mentioned area.

The upper 15 ft. of the dam has been assumed to offer no resistance as an arch, on account of the openings for wasteways. This portion of the dam does, of course, act as an arch to a certain extent, but it was considered better to disregard it entirely, the error, if any, being on the side of safety.

The deflections at the crown of the arch only were considered. This is not strictly correct, as the deflections of the vertical beams are not reduced in the same proportion as the deflections of the arches, as we approach the abutments. Fig. 10 shows the change in deflections for the arch at crest of dam and for the vertical beams at different points on the dam. It will be noticed that the arch deflections decrease more rapidly than the beam deflections on one side of center while the reverse is true for the other side.

A computation was made to determine the stresses in the dam considered as an arch, the resistance of the vertical beams being disregarded.

The following table shows the axial pressure per square foot for different depths on the arch, reservoir full:

Depth, ft.	$\frac{P R}{1}$
40	15.5
80	19.9
120	22.4
160	24.2
200	25.7

It will be seen from this table that if the vertical beams offered no resistance to overturning the arches alone would be able to resist the pressure of the water without developing excessive axial stresses.

TEMPERATURE STRESSES.—In computing the arch deflections due to change of temperature, the dam was considered an arch fixed at the ends, and the deflections computed from the moments due to the horizontal thrust at abutments, by the formula,

$$D = \Sigma \frac{aMd}{EI}$$

M being the bending moments taken at successive equidistant points on the arch.

d the distance apart of the points at which moments are taken.

a the distance from the point whose deflection is required to the point at which moment is taken.

In the case of a rise in the temperature of the dam, when the reservoir is partly filled, the deflection up-stream of the arch at and near the crest of dam causes heavy bending moments in the vertical beams; these moments are intensified by the pressure of the water tending to bend the lower part of the dam down stream.

An attempt has been made to determine these moments. It is impossible to determine them accurately, but the following analysis is probably a fair approximation.

By referring to Fig. 9, the method used, which is practically the same as that used to determine the stresses with reservoir full, will be understood.

The reservoir is assumed filled to a point 100 ft. below crest of dam, and the temperature at crest is assumed 15 degrees above normal, decreasing to normal at 120 ft. below crest. The deflections of the intermediate arch slices between the crest of dam and 120 ft. below crest were not computed, but a curve of deflections was assumed, decreasing from the computed deflection at the crest to zero at 120 ft. below.

The distribution of stresses between the arch and vertical beams is computed for the crown of the arch; the load on the arch, due to the resistance of the vertical beams to the distortion caused by the arch deflection, being assumed uniform for the full length of the arch. As the deflection of the arch decreases toward its abutments, while the beams offer practically the same resistance to distortion for a considerable distance on each side of the crown of the arch, this analysis will give bending moments in the vertical beams, considerably larger than the true moments. On this account a second computation was made, using the arch deflection at a point 15 degrees from crown, this deflection being 0.77 of the crown deflection. The moments in the vertical beams, thus obtained, vary from 75% at 20 ft. depth to 87% at 100 ft. depth, of the moments by the first computation.

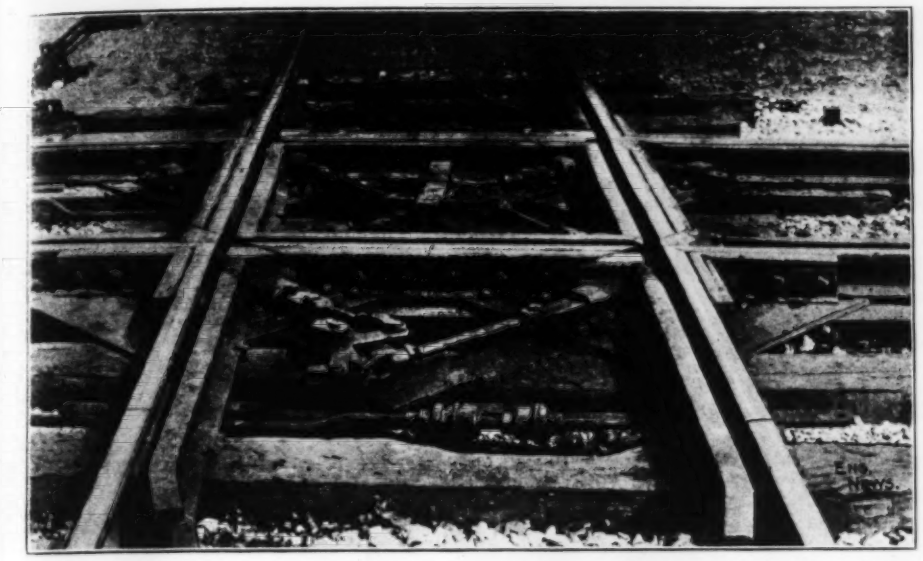
The moments obtained by this second computation are probably nearer the actual moments than those obtained by the first computation; consequently they were used in obtaining the stresses at the face of the dam. The stresses are shown by Fig. 5.

If the down-stream face is reinforced by vertical steel bars, which the writer believes is the intention, the up-stream pressures will be reduced slightly, on account of the change in position of the neutral axis.

THE FILTRATION OF THE CROTON WATER SUPply has again been recommended, this time in a letter or report to the Mayor, written by Health Commissioner Darlington It will be remembered that a like recommendation was made by the special water supply commission about two years ago, and also by the Aqueduct Commission.

CROSSING FROG WITH CONTINUOUS RAILS.

Railway grade crossings are objectionable features from the standpoint of the maintenance-of-way and mechanical departments owing to the breaks in the rails causing severe shocks, which are injurious to the wheels and serve to rack the track at the crossing, besides causing severe wear of the rails. Numerous attempts have been made to devise a crossing with movable parts which will give a continuous rail to the track which is being used, but none of these devices have had more than an extremely limited application, and most of them have proved defective through complexity of parts and inability to stand the severe conditions of service.



GRADE CROSSING WITH CONTINUOUS RAIL.
Morden Frog & Crossing Works, Chicago, Builders.

With a successful device of this kind in connection with crossings protected by interlocking plants, the rail could be automatically made continuous for whichever track is given a clear signal. Such an arrangement is now in service in the crossing of the Indiana Harbor Ry. and the East Chicago Belt Ry., at Graselli, Ind., and the device promises to fulfil the requirements. This crossing has been in service for eight months, including the whole of the past winter, under comparatively heavy traffic. Both lines have a belt line business, and the Indiana Harbor Ry. in addition has 11 suburban trains daily, the average daily movements being 33, representing some 2,000 wheels.

The crossing is provided with a substantial foundation and has maintained good surface and line with very little attention. The only perceptible wear is in the rail head; the movable parts showing no wear whatever. This is especially noticeable with the movable blocks at each angle, which apparently come to as close a bearing as when first installed. The attention required at this crossing during snow storms is found to be equivalent to that required at a split switch. One special feature is that all the operating parts are standard parts of interlocking apparatus, so that the cost of construction is much less than if special parts had to be made.

It will be seen from the accompanying view that four grooved rail blocks move diagonally in the corners of the crossing, being so connected and their movements so adjusted that a continuous running rail may be given either route. A locking device is used, consisting of simple plunger rods operated together and acting through substantial guides; this engages with slots in each rail block in both the open and closed positions. The rail blocks are of cast steel with approximately the same carbon content as the rail steel. Two additional levers are required for operating the crossing, one for the movable rail blocks and one for the lock, these levers being so interlocked with the switch and signal levers that the continuous rail can be given only for the route for which the switches and signals have been cleared.

This device is the joint invention of Mr. Geo. B. Leighton, Vice-President of the American Steel Casting Co., and Mr. Chas. Hansel, M. Am. Soc. C. E., who has for several years been prominent as a signal engineer and manufacturer, and has developed the invention to its present practical condition. It is made by the Morden Frog & Crossing Works, Rookery Building, Chicago, which has the sole right to its manufacture in the Western States, and is now building a few for use on different railways.

LEADED JOINTS FOR LAP-WELDED STEEL AND WROUGHT IRON PIPE.

By Edward Prince,* M. Am. Soc. C. E.

As large-sized lap-welded steel pipe is now made in thirty-foot lengths and more, and of an inside diameter of 20 ins. and upwards, engineers are naturally casting about for the best circumferential joint which can be made. There is considerable literature on the subject, but probably the best article which has been published is the one by Mr. D. J. Russell Duncan, Assoc. M. Inst. C. E., M. Inst. M. E., Managing Director of the Steel Pipe Company, Ltd., Kirkaldy, Scotland. (See Engineering News, April 25, 1891; also the original, in Proc. Am. Water-Works Assoc., 1891.)

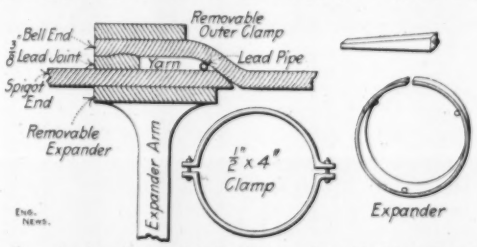


Fig. 1. Leaded Joint for 24-in. Lap-Welded Steel Pipe, with Expander and Clamp for Making Same.

Fig. 1 is a sketch of a lead joint, designed for a lap-welded pipe 24 ins. inside diameter and ½-in. thick, with ⅜-in. of lead. The expander and the outer clamp can, of course, be made in a variety of ways other than that shown. The object of both these appliances is to hold the pipe very firmly while being yarned and calked, which is especially necessary when the pipe is, say, 3-16 or ¼-in. thick. This pipe joint allows of some deflection and absolutely does away with rivets. A mild steel pipe permits calking on the inside, if the diameter is 20 ins. or upwards, by which means the whole interior of the pipe becomes true and smooth.

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I have found that a small, light lead pipe, soldered at the ends and inserted in the joint, driven well back and calked well in place, makes a good seat for upsetting or calking the spigot end, and also holds the yarn from being inadvertently driven into the inside of the pipe.

The best cement I have found to put with the yarn to make the joint water tight against high pressure, is dry Portland or any good hydraulic cement. From the moisture it soon becomes saturated and sets, becomes like stone and lasts indefinitely, and besides it is much cheaper than anything else I have tried and makes a good backing for the lead ring in calking.

I do not know that it is necessary, but with high pressures, such as 500 to 1,500 lbs. per sq. in., I have recommended a little recess, as shown, in the bell, and have always chipped and calked the bell back upon the lead.

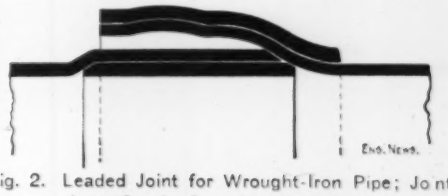


Fig. 2. Leaded Joint for Wrought-Iron Pipe; Joint Iron Stock Co., Upper Silesia.

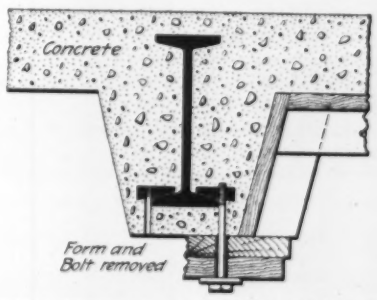
For light pipe, say, ¼ or ⅜-in. thickness, of large diameter, the clamp for the bell and the inside expander ring, both simple in construction and easily removed to the next joint, will be found to be very inexpensive to make and use, and at the same time very useful.

Fig. 2 is reproduced from a drawing of a lap-welded wrought-iron pipe, made at Zawodzie, near Kattowitz, in Upper Silesia, by the Joint Stock Iron Co., formerly Rhein & Co. This shows a considerable advance in joint design, although it is evident that, except for purposes of strengthening against outside or crushing pressures, the joint does not need permanent reinforcing bands inside and out.

The company just named is at the head of the world, so far as I can ascertain, in the manufacture of welded iron and steel pipe by the process known as the pure hydrogen jet; that is, water, gas, and air. The jet is free from sulphur and other things injurious to the metal, and enables the manufacturers to guarantee 95% of the strength of the sheet. The pipes are welded together on the lateral seam to a length of 6 m. (about 20 ft.), and if over that length is required, the joints are welded together on a circular welding bench. By this process sections of pipe have been constructed 146 m. (about 480 ft.) in length, and with diameters ranging from 3 m. (9.84 ft.) to 4 mm. (0.156-in.).

REMOVABLE HANGER FOR CONCRETE FLOOR FORMS.

In constructing haunched concrete floors between steel beams the usual practice is to hang the floor forms from the beams by means of hook bolts catching onto the lower flanges of the beams. This practice ensures a perfectly safe support for the forms, but is objectionable in the respect that the hook bolts have to be sawed off flush with the concrete, thus destroying their value and risking the danger of rust stains. A form of hanger which has recently been put on the market by Mr. J. H. Dousman, civil engineer, of Kansas City,



Removable Hanger for Concrete Floor Beam Forms.

Mo., avoids these difficulties in the manner shown by the accompanying illustration. By greasing or soaping the hanger bolt it is readily screwed out of the concrete, leaving the cast-iron hook head behind. The hole that remains is filled with mortar. The only loss of material in using this hanger is the hook heads, which are left in the concrete. These heads for ½-in. bolts suitable for 6 to 18 in. beams, cost f. o. b. Kansas City, $35 per thousand, and this is considerably more than offset by saving the labor of sawing off the old form hook bolts.

THE "LITTLE JAP" PNEUMATIC DRILL.

One of our contemporaries in the mining field recently published an editorial predicting that in a few years the large air or steam drill will be as obsolete as the log cabin. Such a statement shows a very defective knowledge of rock drilling conditions, for there always will be innumerable places where large and powerful drilling machines will be the most economical. In open cuts, for example, deep holes, spaced as far apart as possible, are necessary to secure a low cost of drilling *per cubic yard* of rock excavation. Our contemporary had evidently confused costs of drilling per lineal foot with costs of drilling per cubic yard, and was thus led to make its singular prediction. However, there are many places where wide spacing of drill holes is impracticable or undesirable, and where holes of comparatively shallow depth will serve. For example, in driving tunnels, in stoping, in block-holing, in plug-hole drilling, and the like, there is usually no need of deep holes spaced far apart. In such cases the type of air drill, now commonly known as the pneumatic plug drill, is rapidly displacing either the larger and older form of air drill, or the slow and expensive hand drill.

Fig. 1. "Little Jap" Drill, No. 2, with Hand Grip.

The pneumatic plug drill, which is an outgrowth of the pneumatic riveting hammer, has recently been developed to a size that necessitates mounting it on a light tripod or column.

Figs. 1, 2 and 3 illustrate the "Little Jap" drill, which is a drill of the pneumatic plug drill type, made by the Ingersoll-Sergeant Drill Co., of New York City.

In this drill the steel remains in contact with the rock and is struck repeated blows by a rapidly moving hammer. The drill is designed along the same general lines of design as the Haeseler pneumatic hammer made by this company. It is distinguished by the use of the Haeseler axial valve, a device which has proved wholly satisfactory in the most severe service of pneumatic tool work.

The "Little Jap" drill is made in three styles. No. 1 is furnished with a hand grip, a finger throttle valve, and an automatic device for rotating the steels. No. 2 has a hand grip also, but a hand lever is used to turn the bit, as shown in Fig. 1. A solid drilling steel is used with the No. 1 drill, but a hollow drill steel is used with the No. 2 drill. A portion of the exhaust air from the No. 2 drill passes through the hollow steel and blows out the chips, cleaning the hole in front of the bit. To prevent the chips from being hurled into the operator's face, a collar is mounted on the cylinder through which a part of the air is discharged against the mouth of the hole. This downward blast of air meets the upward air current carrying the chips, and results in throwing the chips to the side of the hole. Another valuable function of the air current through the hollow drilling steel is that it cools the bit and retains its temper under the severe service of drilling a dry hole.

In the No. 3 "Little Jap," the hand grip is replaced by a back head mounted on the plunger of a long pneumatic cylinder. This cylinder is clamped on a tripod (Fig. 2) or a tunnel column (Fig. 3), and air admitted behind the plunger forces the drill and steel forward against the rock, and holds it there as the hole deepens. The total travel of the plunger is 16 ins., and the complete device constitutes the automatic feed. The drill uses the hollow drill steel with a rose as does the No. 2.

The diameter and maximum depth of drilled by each of these three styles of "Jap" is as follows:

Style.	Diameter.
No. 1	¾-in.
No. 2	1¼-in.
No. 3	1½-in.

At the Homestake Mine, Lead, So. Dakota, are more than 30 "Little Jap" drills at work.

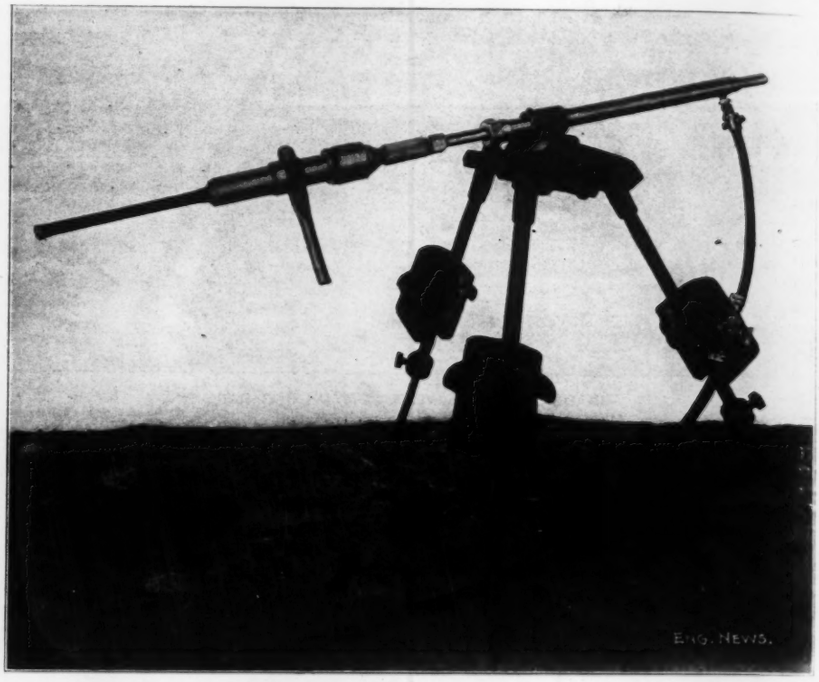


FIG. 2. "LITTLE JAP" DRILL, NO. 3, MOUNTED ON A TRIPOD.

Fig. 3. "Little Jap" Drill, No. 3, Mounted on a Column, as Used for Mine Work.

Jack Haight, the master mechanic, states that in block-hole drilling a No. 2 "Little Jap" averages as follows.

Very hard rock.....	Hole	3 ins. deep, 2 min.
Hard rock	Hole	4 ins. deep, 1 min.
Average rock	Hole	6 ins. deep, 2 min.
Soft rock	Hole	12 ins. deep, 3 min.

In all cases a hole 1¼ ins. diam. was drilled.

The "Little Jap" drill consumes 25 cu. ft. of free air per min., when actually at work drilling.

We are informed by the Ash Grove White Lime Association, Ash Grove, Mo., that "Little Jap" drills (No. 2) are used, and that the average day's work is 75 ft. of hole per drill. The holes are 1¼-ins. diam., and average 2 ft. deep; but holes 5 ft. deep are frequently drilled.

A 260,000,000-GALLON WATER SUPPLY through a 240-mile conduit at a cost of $23,000,000 is the large project announced by the city officials of Los Angeles, Cal. The officials report that options on water rights and on conduit right-of-way have been secured and that the people will soon be asked to vote on the necessary bond issue. The source of the proposed supply is the headwaters of Owens River, on the eastern slope of the Sierras, in Inyo County, Cal. It is said that a concrete conduit 15 ft. wide and 9 ft. high is proposed and that there will be about 17 miles of tunnel. It is expected that the provision of an abundant water supply will lead to the consolidation of outlying municipalities with Los Angeles and in any event contribute materially to the growth of Los Angeles and vicinity. Mr. Wm. Mulholland is superintendent of the water-works of Los Angeles and Mr. Owen McAleer is mayor. In development of this water supply scheme an important part has been taken by Mr. Fred Eaton, M. Am. Soc. C. E., who has served as both city engineer and mayor of Los Angeles. Brief comment on this water supply project may be found in our editorial columns.

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A notable water-works project for the better supply of Los Angeles is outlined in a note elsewhere in this issue. Never before, so far as we know, has an American city proposed to go so far afield (240 miles) for water, and with the exception of the pipe line for the supply of the Coolgardie goldfields of Western Australia we do not know of so long a water supply conduit anywhere in the world. The Coolgardie pipe line is only 30 ins. in diameter, but it is about 330 miles in length. (Eng. News, Feb. 10, Oct. 13 and Dec. 29, 1898.) Instead of being a gravity line water is pumped through it to an elevation of 1,313 ft. This lift is distributed between nine pumping stations, and the nominal daily capacity of the conduit and pumps is 5,000,000 Imp. or 6,000,000 U. S. gallons.

The causes for going so far for water in Western Australia and in California are in some respects similar but differ materially in that at Los Angeles there is a highly developed and prosperous community, instead of a series of gold mining camps. For years Los Angeles has obtained close at hand water enough to supply its most pressing needs, at least, but its future and that of both town and country around it depend upon a large increase in the water supply. The near-by sources are not sufficient for the prospective wants of the city and its suburbs and are more than needed for irrigation. If the future of the city will warrant so large an outlay, and that future certainly looks bright, then Los Angeles is to be congratulated on having at last solved a water problem which has been more perplexing and serious than outsiders have suspected and than most of its own citizens, even, yet fully realize. A further cause for congratulation of which we gladly avail ourselves is that when the city took back the water-works from a private company, a few years ago, it had the good sense to retain the services of the superintendent who had ably served one of those companies for many years.

A unique feature of the recent appraisals of the water-works of Waterville and Augusta, Me., was the introduction of expert testimony on the relation between the character of the water supplies and the typhoid epidemics in the Kennebec Valley during 1902-3. A paper based on facts collected for this testimony appears in the Journal of the New England Water-Works Association for June, 1905. This is not the first time, by any means, that the character of a water supply has been an element in appraisal proceedings, but we do not remember a previous instance of the introduction of typhoid statistics. This typhoid investigation is all the more notable because of its thoroughness and of the number of municipalities covered. It is also significant in that it is in line with a growing tendency to hold purveyors of water financially responsible for ill results from impure supplies. It is very difficult to do this through the agency of suits for damages brought by suffering water consumers or their surviving friends, but in the case of privately owned works there may be a day of reckoning when the works are taken over by the municipality, as sooner or later most private water-works are. In the Maine cases the proximity of the typhoid epidemics to the appraisal proceedings must have given additional force to the arguments that the dangerous character of the supplies lessened the value of the works. Just what weight the appraisers attached to this and other evidence on the poor quality of the water supplies will probably never be known, as it is the custom of appraisers to state their awards in lump sums. In the first of the Maine water district cases the appraisers were instructed by the State Supreme Court, on requests from both sides, what to include in or exclude from their valuations. It was stipulated by the court that "the quality of the water furnished" was to be taken into account. In future appraisals it is to be expected that much weight will also be given to the character of the water being supplied. It should be added, in conclusion, that the paper on which our principle comments have been based was presented by Messrs. Geo. C. Whipple, Assoc. M. Am. Soc. C. E., and Dr. E. C. Levy, and that it was preceded by a general account of "The Maine Water Districts and Appraisals," by Mr. Harvey D. Eaton.

The outbreak of yellow fever at New Orleans is a bitter but much-needed medicine for a country that has been congratulating itself and receiving the congratulations of others for having stamped out the disease in Havana and elsewhere in Cuba. The discredit for the outbreak, however, cannot be charged against the United States as a whole, in its official or unofficial capacity, for the deplorable and evidently needless incident appears to have been due to local causes. It is not for us to say, particularly with the information now at hand, that this heavy blow upon New Orleans and a large portion of the South is due to the inefficiency and neglect of the local health service, but we think there will be no injustice in saying that a more enlightened public opinion would have enabled the local authorities so to fortify themselves against an invasion of the dreaded disease as to have made a serious outbreak impossible. For what Major Ross says about preventing malaria, in an article abstracted eleswhere in this issue, is applicable in a general way to yellow fever. In fact, it will be noted that at the conclusion of his article Major Ross holds up what the United States did with yellow fever in Cuba as a shining light by which to condemn Great Britain for neglecting to work similar marvels in the elimination of malaria in its dependencies. It appears, however, that the general public at New Orleans, or, to put it more strongly, the more enlightened portion of it which presumably rules the city, was not ready, prior to the outbreak, to enter upon a thorough-going campaign of eliminating mosquito breeding places by drainage, oiling, screening (in the case of cisterns and the like), and also by screening the doors and windows of houses. Those were the measures that rid Havana of yellow fever after centuries of suffering and those are the means that may be employed to prevent yellow fever throughout the South and malaria everywhere. If a small part of the money and the mental energy that is now being spent in quarantining other parts of the South against New Orleans were expended in rational health measures against mosquitos the South would be ensured of immunity from yellow fever and be vastly benefited in general healthfulness as well. North and South alike must awake to the grave menace of its poor and ignorant foreign populations of various nationalities as foci of communicable disease. Most of the yellow fever of New Orleans, we understand, has been in the crowded Italian quarters. Under present conditions of overcrowding, uncleanliness and ignorance, Italians and foreigners of other nationalities in many of our cities, and even in relatively small towns, are very hotbeds of infection. The evil would be bad enough if infectious diseases and high mortalities were confined to these classes, but in one capacity or another large numbers of these people circulate among and serve the rest of the population, spreading disease and death wherever they go. It is imperative, therefore, that health authorities keep a close watch of these dangerous elements and do all that is possible to improve their sanitary environment and to increase their very limited knowledge of the laws of public and private health. At the same time, health authorities and the enlightened portion of the community should recognize and employ to the utmost the beneficent possibilities of modern engineering and sanitation.

COLLISIONS BETWEEN VESSELS AND DRAWBRIDGES.

Movable bridges are not, primarily, intended to be bumping-blocks for large ships, and bridge engineers rarely if ever plan their designs with such heavy service in view. Two recent accidents noted in our columns illustrate the effects which may be produced by a collision between a steamer and a swing-bridge, and suggest a consideration of such occurrences by bridge designers. Less than a month ago a steamer struck a swing bridge at Milwaukee, Wis., and damaged it seriously, besides coming within a hair's breadth of throwing a street-car into the river (Eng. News, Aug. 3, 1905). A month before that, an occurrence of precisely the same kind took place at Houghton, Mich. (Eng. News, June 1, 1905), pushing the bridge bodily off its supports and throwing it over into the water. If we were to judge by the past two months alone, it would seem that collisions of this sort are not such remote possibilities that designers of bridges need not concern themselves with protective provisions.

River transportation through a series of one or more drawbridges may be compared with railway traffic under a block system. The vessel, like the train, moves past a succession of block-points at each of which it may find a signal directing a stop. The vessel moves much more slowly than the railway train, but also it starts and stops much more slowly; it lacks the ability to brake, except by reversing, a means sometimes involving risk or difficulty. The signals at the blocks also differ; the signal for the vessel is obtained from the obstacle itself, the bridge. This signal, moreover, is peculiar. It is always at danger except during a short period while the vessel is passing; as the vessel approaches, the signal theoretically should clear, but there is no absolute certainty of this; it may clear and it may not. The vessel is thus continually running at Caution, prepared to stop, even when close in front of the signal. Safe operation under such conditions requires prompt service and mutual interaction on the part of the pilot and the engineer of the vessel and the bridge-tender.

It will be admitted that opportunity for accident exists in this combination. The possibility is greatest in narrow river channels, where space for manœuvring is restricted, and especially where bridges are many and close together. This is generally the case in the Lake ports, whose harbors are the banks of narrow rivers running through the busiest sections of the cities and therefore crossed by many bridges.

If collisions between vessel and bridge entailed property damage only, with a certainty that no risk to life were involved, there would, perhaps, be less reason to mention the subject at the present time. But both the Houghton and Milwaukee collisions involved imminent risk of serious loss of life, and the same risk might with reasonable probability be present in any other collision of the sort, at least in the downtown districts of large cities.

Such a probability warrants study of possible means of safeguarding against the danger. Even the risk of mere material damage may in the

case of heavily-traveled bridges be enough to call for such study; the bridges in downtown Chicago, for instance, are so valuable as avenues of traffic that a considerable extra outlay is justified for the purpose of ensuring continuity of their service, or at least minimum interruption in case of accident.

As remarked above, bridges are not built to serve as vessel-stops. In view of the grounds advanced in the preceding paragraphs it may often be advisable, however, to consider them in part with such service in view. This does not necessarily imply that they should be made strong enough to stop a vessel without injury to the bridge structure, but rather that the construction should be such as to minimize the consequences of the accident. On the face of the matter, the most serious risk to life lies in the complete demolition of the bridge, as when the collision causes the bridge to fall. The same condition entails the maximum interruption to traffic, since it blocks both river and street traffic, and requires clearing the wreckage and putting new superstructure in place. Contrariwise, a collision is rendered less serious in its consequences if the bridge is able to hold up its own weight even when an accident destroys a main truss member, or even two members in a single panel of one truss. Means for assuring this function in the case of swing-bridges on busy streets and over busy rivers should in many cases be well worth their cost.

Bridges as usually constructed are not capable (in theory) of holding up after a main truss member has been ruptured. In practice they sometimes prove to possess this ability. About a year ago, for example, one of our correspondents described a bridge accident where the lower chord of one truss in a two-truss bridge was broken without the bridge falling. The stringers, framed into the floor-beams, doubtless carried the tension given up by the broken chord. But this reserve capacity, which is not provided for in the designing or detailing, can hardly be relied upon in the general case. Our correspondent therefore made the argument that important bridges, especially double-track railway bridges, should always be designed with three trusses in place of two. This recommendation may be accepted without reserve for the case of swing-bridges here considered. Bridges in the locations referred to are always of considerable width, usually carry two street-car tracks, and are designed for a heavy class of highway loading; the adoption of a three-truss design would therefore entail difficulty but rarely, and generally only slight increase of cost.

A good primary protection against collisions may be contrived, in the case of highway bridges, by suitably utilizing the sidewalk floor. In the typical construction, with sidewalks outside of the trusses, a colliding vessel must cut through the sidewalk before reaching vital parts of the bridge. With the simple plank sidewalk, the blow of the collision finds little resistance before reaching the truss. Yet, if the sidewalk be of suitable strength, the roadway-plate as a whole, with its great lateral stiffness, offers an excellent means of receiving and distributing the blow of a collision. A heavy concrete sidewalk properly stayed to the roadway floor should therefore be a good investment, provided the roadway itself be of solid construction. In other cases one or two lines of heavy timbers under the outer edge of the sidewalk may be of service.

To give reserve strength in case the lower chord is ruptured—a rather probable injury in collision—use may be made of the floor-stringers, to carry tension in case of accident. By improving the usual framed connection between stringers and floor-beams, and giving the laterals efficient connection to the stringers, considerable assurance may be secured against collapse of the bridge when a lower chord panel and its adjacent laterals are torn away. But this suggestion illustrates the care with which the entire subject must be considered. The continuity of the stringers is, ordinarily, rather objected to than sought for. It has been shown by extensometer measurements, we believe, that railway bridge stringers do carry part of the general bottom-chord tension, and that the floor-beams near the ends of the bridge are subjected to transverse bending by this cause. These effects imply secondary stresses, whose worst feature usually is that they are greatest at the weakest points. If this continuity is fully considered at the time of design, means can no doubt be contrived to stay within the limits of prescribed safe stress and avoid secondary effects. Under present types of construction reliance on stringer tension is hazardous.

The discussion of means for dealing with vessel-collisions in bridge design is, of course, outside the scope of these paragraphs. We desire to direct attention to the two collisions recently recorded in our pages, and to indicate that something may be done by bridge designers to minimize the effects of similar accidents. The details of the matter depend largely upon a study of effects observed in these and similar accidents, effects which are but partly represented in the two cases here mentioned. Something of interest may be gathered from each individual bridge wreck. The knocking-out of the end post of the East Water Street bridge in Milwaukee by side pressure of an electric car is, for example, an effect which would hardly have been expected and which warrants reflection in connection with this general subject.

A COMMENDABLE SPECIFICATION FOR SEWER CONSTRUCTION.

Contractors frequently complain that sewer specifications are so indefinite that a sewer contract is more risky than a bet on a horse race. During the past two years we have criticized many of the ambiguous and unfair clauses in specifications, and a review of our columns will disclose the fact that a very large percentage of the specifications criticized were sewer specifications. This does not prove that sewer specifications are worse, as a rule, than specifications for other kinds of work, but it indicates that there are many items in sewer work that are not easily specified. The very fact that all the work is done below the surface of the ground accounts for much of the difficulty not only in doing the work but in specifying how it shall be done.

Nevertheless, the more frequent use of boring tools to determine the nature of excavations, and the exercise of care in drawing specifications so as to take the burden of uncertainties from the contractor's shoulders, will eventually make sewer contracting almost as safe a business as pavement contracting. That many city engineers are anxious to accomplish this result is apparent from such letters as the following from a city engineer:

> Your criticism and advice regarding contracts has always been of great interest to me, and has often been of assistance in framing special clauses for our work. We are now getting up a new form of general contract for sewer construction, and I send herewith a proof of this new form, practically finished, but not yet gone to press. When printed, the sheets will be 8 × 9 ins. to fit our files, with a margin which can be trimmed to standard 6 × 9 size. If you could find opportunity in the near future to review this in the "News," you would confer a favor upon us engineers, and by doing this before the edition is finally printed, your suggestions may be adopted and be of service to the contractors whose interests I beg to assure you, we have really tried to consider.

This closing sentence shows a spirit which we believe to be the prevailing spirit among engineers, but a spirit that some contractors have been inclined to regard as non-existent. Contractors, we think, have usually failed to make known their point of view about specifications, until *after* the award of the contracts, and then not until the specifications have begun to pinch their pocket books. We believe there are few engineers who would not listen with patience, and even with eagerness, to any contractor who has a well thought out criticism to make regarding any specification clause, provided the criticism is made by a contractor who is not actually engaged in doing work under that specification.

Taking the specification that has just been submitted, we find an opening clause that may be unobjectionable in this particular contract, but one that might lead to trouble where contracts are awarded under other ordinances. We quote:

> Bidders will state, both in writing and in figures, the proposed price for each separate item of the work called for in the annexed blank, by which the price will be compared. If any price is omitted the blank may be filled with the highest price named by any bidder for that item or the proposal may be rejected.

It is the last sentence that seems to us of doubtful validity. Under many city ordinances contracts must be awarded to "the lowest responsible bidder." Now, if a bidder fails to bid on one item, and the city engineer afterward finds in that item with the *highest* price bid by a competitor, is the contract one that is awarded to the *lowest* responsible bidder? Again, might not a tricky contractor use this means of escaping the acceptance of a contract if he were to find his total bid much below his nearest competitor's? Would the courts hold him to signing a contract in which he had apparently erred by omitting to bid on one item? We believe that no contractor could be compelled to perform a contract, or to forfeit his certified check, if he could show that he had failed to name a price on one of the items.

Passing to another clause:

> The number of working days named in the proposal will be estimated in the comparisons of bids at the rate of $5 per day.

A clause like this should never be omitted where bidders are called upon to name the time of completion.

As illustrating the care with which the detailed methods of construction are specified, the following quotation will serve:

> Only one man under each inspector shall be allowed to lay tiles on the main sewer, and only one on house connections; and no tile shall be laid except in the presence and according to the directions of the Inspector.
>
> Tile pipes shall be so laid as to be evenly supported throughout the whole length of the barrel, with no weight resting on the bell. If the trench is dug deeper than the grade of the barrel no spalls, shims, or lumps shall be used to raise the pipe to grade, but an even bed shall be formed of sand or approved fine material properly tamped.
>
> Joints shall be made as follows: (1) Line with mortar the lower third of the entire circumference of the bell; (2) insert the pipe to be laid and a jute gasket freshly dipped in neat grout; (3) bring the pipe to grade and line; (4) caulk the gasket tightly into the joint; (5) fill the joint with mortar mixed rather stiff, using a rubber mitten; (6) tamp mortar into joint with an approved tool until it is solidly filled; (7) smooth on mortar, using a rubber mitten, until its surface makes a bevel of not more than 60 degrees with the pipe; (8) protect the cement (preferably with burlap), and fill around the barrel of pipe with sand or similar material, tamping solid with an approved tool; (9) remove burlap and cover joint and pipe with fine material; (10) clean out and point joint on inside of pipe. On pipes less than 12 ins. in diameter cleaning out may be done with a bag, stuffed so as to tightly fill the tile, drawn through each length of tile as it is laid. All pipes must have the lower third of the joint completely filled with cement; pipes larger than 18 ins. shall have the lower half pointed and larger than 24 ins. the whole of the joint, using neat cement on the top half if necessary.

The clauses relating to payment have been drawn with particular care, so that the contractor may feel sure that he will not be required to do something for nothing. For example, it is provided that for all surplus material hauled more than 1,000 ft. an ovehaul price is to be paid per cubic yard for 100 ft. of overhaul. It is apparent that the engineer has put on a contractor's spectacles in preparing these specifications. He may rest assured that by so doing he will get closer prices in the long run, and will obviate many a claim for "extras."

LETTERS TO THE EDITOR.

A Question in Compound Engines.

Sir: Will some one of your readers who is a mechanical expert give me a solution of the following question?

A compound pumping engine, using steam under 125 lbs. gage pressure, has a high pressure cylinder of 8 ins. and a low pressure cylinder of 12 ins. diameter, has a slide valve and exhausts into the atmosphere. What is the M. E. P.? Several formulas and rules which I have give widely divergent results, hence the question.

Subscriber.

Salem, Va., July 28, 1905.

Low Pulling Early Stage Portland Cement.

Sir: Referring to the letters of Mr. Bernard Enright and Mr. F. H. Lewis in your issue of Aug. 3, on low pulling early stage Portland cement, a paper submitted by myself at the Atlantic City meeting of the American Society for Testing Materials, the following comments seem appropriate.

The good final showing of the tests on "Rejected" materials in my tables loses much of its weight when my remarks thereon are noted; namely, that this cement represented "comparatively a very small proportion of the total tested," and "would indicate that the grading of material by its low tensile strength at seven days, irrespective of any specified gain, would result (everything else being normal) in better product than that high early stage strength cement so generally met with."

The manufacturing contracting company for the cement used on the New York Rapid Transit work, and myself, have always agreed that while it is possible under our specification and system of inspection that some good material may have been rejected, viz., this very small percentage of the whole, whose records your two correspondents make too much of in comparison with the large volume of records on the balance of material, we are both quite sure no bad cement has been accepted; though more or less of this accepted material has in some instances closely approached the actual minimum limit of our requirements, and showed up satisfactorily in the long run, quite contrary to Mr. Lewis' assertion, that such material is not as good as the higher pulling cement.

No one is readier to admit than myself that tensile tests are not the ideal ones, especially for long time results, where the crystalline character of the formed product of the briquettes tested may very easily influence results. For that very reason the comparatively few tests on our rejected material, which on their face at the last period show slightly better than the accepted material record should have very little weight against the infinitely larger number making up these latter records, which do show best at all periods, except the very last three years, and then with only a difference of 28 lbs. in favor of rejected material. It is certainly not my opinion that as stated by Mr. Lewis the better material attains its strength comparatively early; this retrogrades as shown.

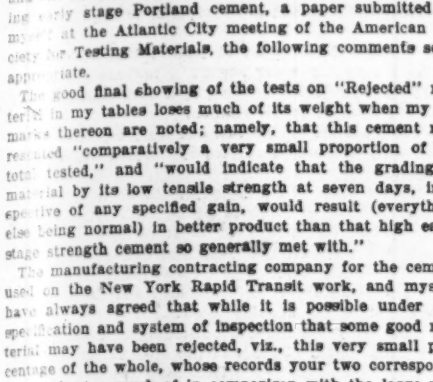
The Board of Rapid Transit Railroad Commissioners of the City of New York through its Department of Inspection of Material, acting for its Chief Engineer, certainly see no reason to warrant the slightest recession from their original position, "That low pulling early stage Portland cement, showing a minimum gain in both neat and sand between seven days and twenty-eight days," is the best material.

Very truly yours,
W. A. Aiken,
General Inspector of Material.

Bethlehem, Pa., Aug. 4, 1905.

A New Style of Roof: Solid Wooden Slab Construction Without Purlins.

Sir: I beg to call attention to a method of roof construction for mill and factory buildings that I have been using since 1903 with excellent results, which I believe to be a novel application of an ordinary construction. It consists in spanning between the roof trusses or rafters with planks laid on edge and nailed together solid, forming a wooden slab from truss to truss and doing away with the purlins altogether. For 16-ft. bays I have been using 2 × 4-in. hemlock, surfaced on two sides to 1¾ × 3¾-ins., as shown in accompanying sketch.


Roof Covering (Slate, Tarb Gravel, Etc.)
1¾"×3¾" Hemlock Planking, nailed together solid, breaking joints every 12" in length of Rafter.
Ridge.
Rafter or Roof Truss 16'-0" C. to C.

Solid Wooden Slab Roof Without Purlins.

A roof built thus in 1903 for a mill building located in the northern part of New York State, where the snow loads are very heavy, has proved very satisfactory, and I have recently used the same construction for several large new buildings for the same company. In designing the steel frame of the Carnegie Library extension at Pittsburg, an elaborate structure some 400 × 600 ft. in plan, for Messrs. Alden & Harlow, architects, I used 2 × 6-in. yellow pine for the roof planking spanning up to 16 ft. The roof covering in this case was a heavy tile, and no deflection whatever was allowable.

The advantages of this construction may be summed up as follows:

(1) Cheapness. All purlins are eliminated; the planking may be of the cheapest wood, is prepared in simple form with no waste in section or lengths, and does not require skilled labor in laying (excepting, of course, where labor unions are in control of operations).

(2) Fire resisting qualities. The upper and lower surfaces offer to the flames a wide unbroken area of practically solid wood, which flames of ordinary intensity could not take hold of.

(3) Non-conductivity as to heat and cold.

(4) A permanently smooth and neat under surface, there being no tendency of individual planks to warp as is the case with tongued and grooved planking.

(5) An unbroken surface for the economical spacing of the automatic sprinklers.

Yours very truly, Chas. Worthington.

134 W. 70th St., New York, N. Y., August 3, 1905.

A Diagram for Computing the Power-Capacity of Belts.

Sir: The accompanying chart gives a graphical representation of the various conditions met with in belt drives, and provides a means of comparing clearly the different results due to centrifugal force at high velocities, as well as showing the importance of getting as great an arc of contact as possible.

The working stress of belt per square inch, or the safe working tension in the tight side, T_n, is represented by the outer concentric circle marked 250. The tension in the slack side, T_0, per square inch of belt section, for different velocities and arcs of contact, is represented by the distances from this outer circle, measured radially, to the velocity curves. Since the available working pull is the difference in the two tensions, the distance from the center of the concentric circles to these velocity curves, taken on a radial line corresponding to the arc of contact, gives the transmitting force per square inch of belt.

The chart is based on the formula:

$$T_n = 10^{.00758 f a (1 - z)} T_0$$

a well-known formula in which centrifugal force is introduced.

T_n = coefficient of friction, taken as 0.30,
T_0 = tension in slack side.
f = tension in tight side.
a = arc of contact in degrees.

$z = \frac{WV^2}{9{,}660\,T}$, in which

W = weight of leather belting per cubic inch, or 0.035 lbs.
V = velocity of belt in feet per minute.
T = working tension in pounds per square inch (250 lbs.).

Example: What horse-power will a 42 × 16-in. pulley at 200 revolutions per minute, with arc of contact of 135°, deliver when the belt is ⅜-in. thick?

The belt speed is about 2,200 ft. per minute, and for the arc of contact 135°, the transmitting force per square inch according to chart is 120 pounds.

16 × ⅜=6 sq. ins., which at 125 lbs. per sq. in. gives 720 lbs. pull.

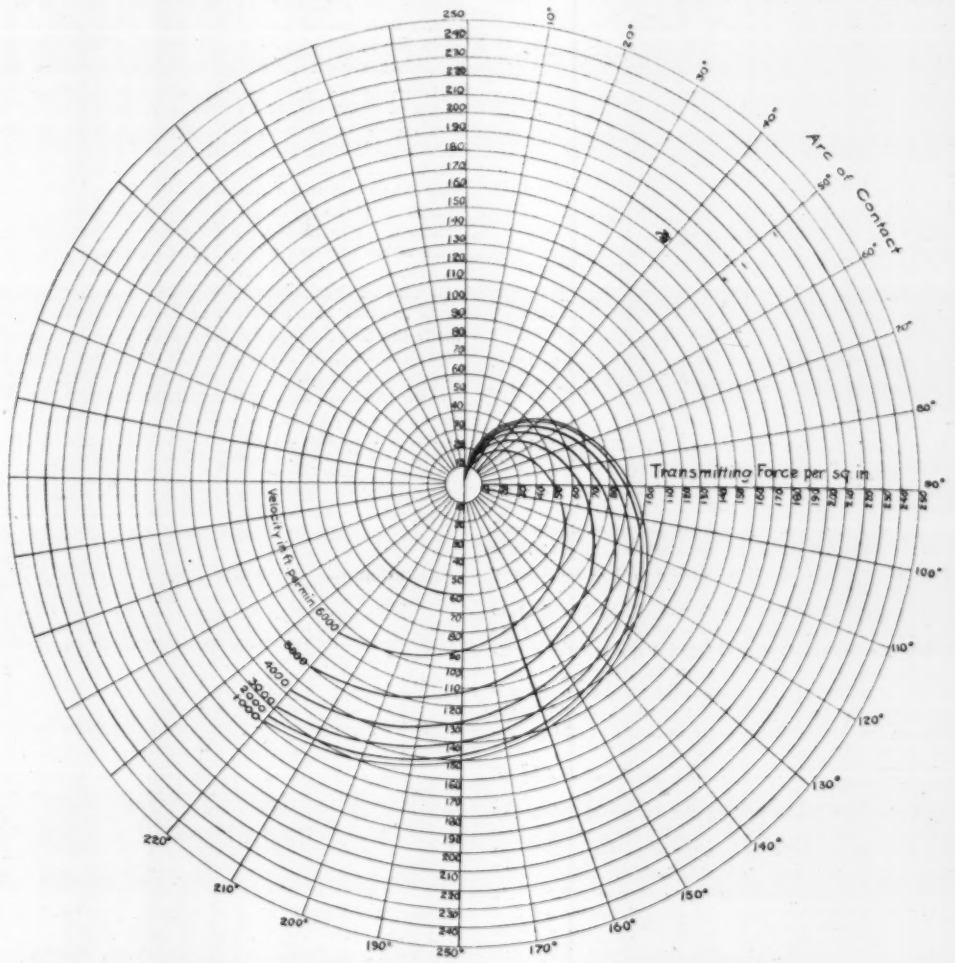
$$\frac{720 \times 2{,}200}{33{,}000} = 48 \text{ HP.}$$

The horse-power for a 180° arc is about 25% more than for a 130° arc of contact, and a belt will only transmit about 60% as much at 6,000 ft. per minute as at 1,000 ft. per minute.

Single leather belting is from .22 to .25 thick. Double leather belting is from .25 to .35 thick.

Yours sincerely, R. W. Dull.

St. Louis, Mo., July 31, 1905.



A DIAGRAM FOR COMPUTING THE POWER CAPACITY OF BELTS.

Establishing Irrigation Canal Tangents So Cut and Fill Will Balance.

Sir: In your issue of July 6, 1905, "Tangent" asks for help in the solution of the problem of balancing cut and fill on irrigation canal tangents.

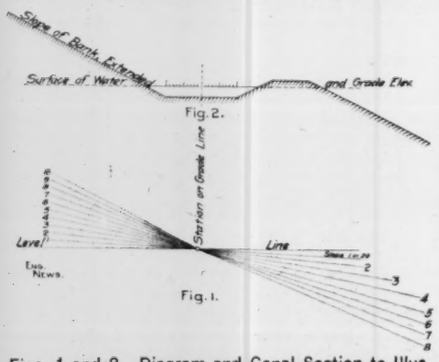
The problem that he has set for himself is so difficult of absolute solution as to suggest that the easiest way out of the difficulty would be a complete change of plan of laying out the work.

While there are some features of a tangential canal, for conveying water for irrigation or other purposes, that are highly desirable, there are so many others in favor of the "contour" canal that in most cases of later practice in the western part of the United States the tangent system of alinement has become nearly obsolete.

The grades usually found in our practice, however, are seldom as great as that indicated by "Tangent" (0.4%), and the velocities due to such a grade would seem very difficult to control at curves between tangents, unless the canal was lined with concrete. Such a grade will produce a velocity so great as to render the maintenance of the wet banks and bottom of the canal to line and grade impossible, except in case of rock or cement lining. I do not know of any kind of soil that would not be moved by a velocity due to such a grade.

There are few cases of late practice where a grade greater than 0.02% is being used, with strong tendency toward reduction of that, in the larger canals.

The cost of excavation is greatly increased by the long hauls necessary in making cuts and fills on tangent constructions, whereas, in the contour form of canal the actual cut can be increased or decreased to balance the



Figs. 1 and 2. Diagram and Canal Section to Illustrate Method of Balancing Cut and Fill on Cross-Sections of Irrigation Canals.

fill required to give proper strength of bank and the length of haul will be of the shortest possible. This feature very much more than makes good for any extra length of the canal by following the contour.

Another feature of this manner of construction and the use of a low grade is the additional area of lands irrigable from the canal, a consideration in most cases not to be neglected.

The method of laying out a "contour" canal is, first, to run a grade line on a slope of say 0.02%, or whatever has been determined as best suited to all the existing conditions, finding and staking the points on the ground, a station apart, and at the elevation of the grade line. This is simply a guide line, the elevation of which should be the same as the surface of the water when the canal is carrying its normal depth.

When this grade line is run the slope stakes may be set as follows:

For guidance in the field in the matter of balance of cut and fill cross sections, construct a diagram with lines drawn thereon for all slopes liable to be encountered along the whole length of canal. Such slope lines on the diagram crossing each other at a common point as in Fig. 1. This common point represents the grade point at any station. On a piece of tracing cloth draw a cross-section of the canal and embankment, with inner and outer slope lines extended as in Fig. 2, and having a horizontal line indicating the surface of the water.

With this tracing superimposed on the other diagram with the water surface line of one coincident with the horizontal line through the grade point of the other, and a little shifting to right or left of the tracing, a balance of cut and fill for any particular side slope of the ground can be arrived at by inspection and the distance that the center line of the canal should be shifted to right or left of the grade stake already set observed by scale on water line of tracing.

In cases where the slope changes much in the width of cut and fill a true profile of such stations should be made and used in lieu of diagram in order to arrive at a very close balance.

Yours, Chas. L. Swain.

Irrigon, Ore., July 17, 1905.

Sir: It seems that "Tangent" (Eng. News, July 6) has fallen into the common error of attempting to balance cuts and fills in a canal or ditch line. The primary consideration in most problems in canal location is to place such a percentage of the cross section in excavation that there will be no danger of breaking the banks.

However, assuming that his cross section is such that he is amply safeguarded, and still desires to balance quantities, I might suggest the following method, which I have found to give very satisfactory results:

The primary assumption is that the slope is regular between the outer edges of the embankments, or that an average equivalent has been taken. Given the cross section, a table can be readily prepared showing what depth of cutting at the center for a given slope will call for an equivalent amount of embankment and shrunken excavation. After running the preliminary line, these points of center cutting for various slopes can be taken very rapidly in the field, and plotted as a grade contour on the preliminary map.

This contour is the economical center line, but it will head draws and round points at impossible angles, even though very sharp curvatures are used, which might permit a closer tracing. After projecting the location, there is but one true way to balance quantities, namely, to cross section the line and prepare a mass curve, which will show at what points investigations may profitably be made to avoid waste, borrow, or haul. This curve also limits the investigations to those sections where they are needed, and plainly shows whether or not the ideal location has been found.

I am told that an exposition of the principles of the mass curve has been published in the Engineering News. If this is true, and the editor will kindly supply the date of issue, it will prove of great value to "Tangent." (See Eng. News, Jan. 31 and March 14, 1891.—Ed.)

Very truly yours, V. L. Havens,
Locating Engineer, Farmers' Canal,
Scottsbluff, Neb., July 19, 1905.

Sir: I note the inquiry of your correspondent from Canada in the problem of locating canals and irrigating ditches. I beg leave to submit the method which I have frequently used, with satisfactory results. It seems so obvious and simple that it may be in common use, but I have encountered no suggestion of its use anywhere. By this method I have recently located a five-mile ditch upon a hill side, sloping from 3 to 4%. The ditch was partly cut and partly fill, having a wet section of 300 sq. ft. The method is as follows:

(1) First stake the sloping contour throughout the length of line.

(2) Cross section and then plat the ground line on a scale of 10 ft. to 1 in., vertical and horizontal (Fig. 3). Each of these cross-section diagrams, by way of greater convenience, I make upon a single sheet, about 9 by 13 ins. These I have cross-ruled by the printer in inches and tenths. Each sheet is numbered plainly with the station number, and shows on the cross section the point of the stake on the ground and the datum elevation of this stake. This manner of recording the sections diagrams gives a portfolio of flat sheets for ready convenient handling and is far superior to the continuous roll.

(3) Compute the elevation at each cross-section stake for the prospective water surface. Indicate this water surface on the sectional drawing by a horizontal blue line across the sheet in proper relation.

We now have the ground line and the prospective water level, which will not be effected by "shying" location of the canal.

(4) Upon a sheet of tracing paper (tissue thin) prepare the cross section of proposed canal, with some definite point, say half way across the water surface, clearly indicated. We will call this point the axis of the canal. This cross section of prospective canal shows slope of cut into the hill-side above the water area and the slope of embankment on the hill-side below.

(5) Now apply the transparent tracing of canal section to the cross-section diagram of the hill-side—seeing that the water lines of each coincide. Slip the tracing back and forth along the water lines until cut and fill seem to balance. The slight error in balance, which the eye will make by this graphic method, will not appreciably effect cost of construction. (The position of the red line tracing is indicated in Fig. 3, at the right of the final cross section.—Ed.)

(6) Note where the axis of this new location falls upon the cross section sheet with relation to the stake already set, and which stake has been duly platted on a map,

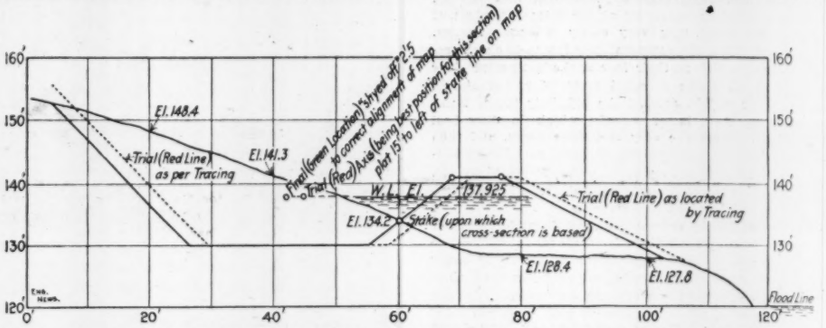


FIG. 3. ANOTHER METHOD OF LOCATING CANAL, SO CUT AND FILL ON CROSS SECTIONS WILL BALANCE.

(The lines indicating tenths of feet have been omitted.—Ed.)

The red location is now the best possible, so far as cut and fill are involved at any particular station, but it is impossible in respect to alinement. The red line suggests a rail fence with here and there the zigzags flattening till they approximate a tangent and here and there crowding the point of a hill or heading a ravine or some small stream approximating a curve.

(7) Draw the final alinement (Fig. 4), conforming as closely as expedient to the red location, cutting out all angles and swinging the curves symmetrically without regard to tangent or radius.

The adherence to mathematical curves and tangents in canal location is the application of railroad practice to wholly different conditions. It greatly increases cost of construction with no appreciable benefit. Canal alinement should follow the contour with such symmmetrical curves as will permit an easy path to the flowing water and produce the least reaction practicable. Where the curves are undesirably short, it is more economical usually to pave or concrete than to construct the heavy work made necessary by a theoretical alinement to tangent and circle. This final location I call the green line, and in a note book record the distance which the axis of this canal will fall to right or to left of the stakes set already on the black line, which is the actual line staked out on the ground.

(8) With the figures of this note book now before me, indicating the distances of the final location to right or to left of the black line, I slip the transparent section of canal to the indicated point upon the cross-section sheet and with a needle prick through the salient angles of cut and fill on each sheet. These points are now readily connected up and each cross section is complete with regard to location of cut and fill. With the same note book in hand walk over the line and reset stakes marked with green to right or to left of the original stakes. The green line, the axis of the canal, is now defined on the

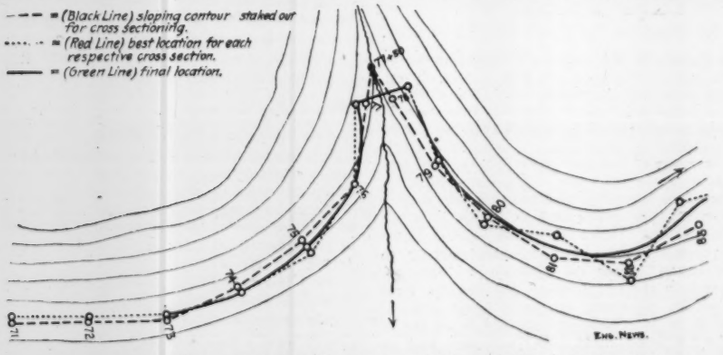

= (Black Line) sloping contour staked out for cross sectioning.
= (Red Line) best location for each respective cross section.
= (Green Line) final location.
Eng. News.

FIG. 4. SKETCH MAP ILLUSTRATING METHOD OF CANAL LOCATION.

ground, and the position of secondary stakes may at any time be placed upon inspection of cross section.

The method does not take into consideration the occasional deep cuts upon some point of hill or the dam across the mouth of some ravine. Each of these must be considered as a separate problem, and be disposed of independently when they occur.

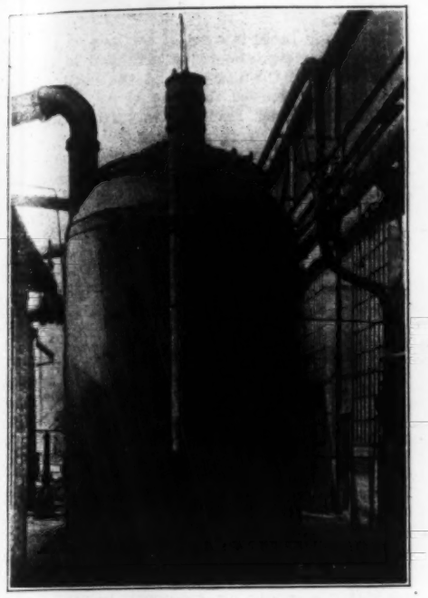


Fig. 1. Rateau Steam Regenerator at the Poensgen Steel Works, Dusseldorf, Germany.

I have here taken considerable space to explain a method which in practice is very simple and effective in application.

Yours truly,

John W. Hays.

Petersburg, Va., July 20, 1905.

Notes and Queries.

Mr. F. A. W. Davis, President of the Indianapolis Water Co., wishes to know whether anyone has succeeded in melting the lead joints of water pipe by means of electric currents.

THE RATEAU REGENERATING SYSTEM FOR UTILIZING EXHAUST STEAM IN LOW-PRESSURE STEAM TURBINES.

A regenerator system for the utilization of exhaust steam in low-pressure steam turbines was described in our issue of Sept. 29, 1904 (pages 282 and 287), and we are enabled to illustrate herewith a plant of this kind which has been in steady use for some months at the Poensgen Steel Works, Dusseldorf, Germany. There are now 23 of these plants in operation or in course of erection in Europe; 15 of these utilize the exhaust from large mine hoisting engines, and 8 utilize the exhaust from rolling mill engines. The system is the invention of Mr. Rateau, inventor of the Rateau steam turbine. It is being introduced into this country by the Rateau Steam Regenerator Co., 226 La Salle St., Chicago, which is contracting for several important plants. The general principles are briefly stated as follows by Mr. M. L. Battu, President of the company:

The Rateau patent steam regenerating system consists in transforming the variable discharges of intermittent-running engines into a constant flow of steam, and then utilizing the regulated exhaust in low-pressure condensing turbines. The regenerator consists of a vertical cylindrical vessel enclosing a succession of cast-iron trays filled with water. The principle upon which the machine is based involves considerations as to the properties possessed by saturated steam and saturated liquids, respectively, and of the heat exchanges which take place between the steam and the water.

Under any determined conditions of pressure and temperature these two fluids (saturated steam on the one hand, and saturated water on the other), composed as they are of similar molecules, preserve a reciprocal condition of equilibrium, such that any variation in the total heat system, which may result from a disturbance of this equilibrium, determines the transformation of either form of fluid into the other form, with a corresponding absorption of heat in the one case and liberation of heat in the other. Given this property, it follows that if the steam which is escaping intermittently be conducted into a receiver containing cast iron and water, the cast iron will absorb heat, upon its surface, and occasion the condensation of a certain quantity of steam, which remains in the state of saturated water.

Owing to this condensation, the steam accumulates whenever an abundant supply arrives in the regenerator, and thus causes a slight elevation both of temperature and of pressure within the apparatus. When the supply of exhaust steam from the engine falls off or ceases, the steady call for steam for the turbine reduces the pressure, and so determines the condition of dis-equilibrium, the necessity for which has already been explained. The latent heat of the steam, held in reserve by the cast iron and the water, serves to vaporize anew a certain weight of water, and the steam is thus regenerated in the form of a constant supply.

These exchanges take place with extreme rapidity, and the variations of temperature and pressure required for the condensation and regeneration of the steam in the interior of the regenerator correspond with slight fluctuations, which can be perfectly controlled by a careful adjustment of the materials contained in the apparatus. The Rateau steam regenerator can be compared to a flywheel for heat, as it stores up heat when the pressure has a tendency to increase and gives back heat when the pressure decreases.

Fig. 1 shows the regenerator at the Poensgen plant, containing the pig iron trays; the cylindrical shell is 12 ft. diameter and 18 ft. high. This regenerator regulates the exhaust of a reversible rolling mill and of a battery of steam hammers, the total discharge of these engines being approximately 22,000 lbs. per hour. Fig. 2 shows the low-pressure turbine (Rateau type) driving two direct-current dynamos at 1,400 revolutions per minute. This turbine receives steam from the regenerator at 15 lbs. absolute and discharges into a central condenser giving a vacuum of 26 ins.

This plant is running at a normal load of 650 electrical HP. measured at the brushes, the steam consumption of the turbine being 33 lbs. per electrical HP., including condensation in the regenerator. The 650 electrical HP. thus obtained by the utilization of exhaust steam, which otherwise would be wasted, is employed by the electric roll train of the mill for lighting power and by an electric furnace. The cost for maintenance is nothing for the regenerator, and is extremely low for the turbine.

In Fig. 3 the upper irregular curve shows the variation in volume of steam from the primary machines (rolling mill engines and steam hammers) and the lower curve shows the practically uniform pressure maintained in the accumulator or regenerator. This record is for one hour's run.

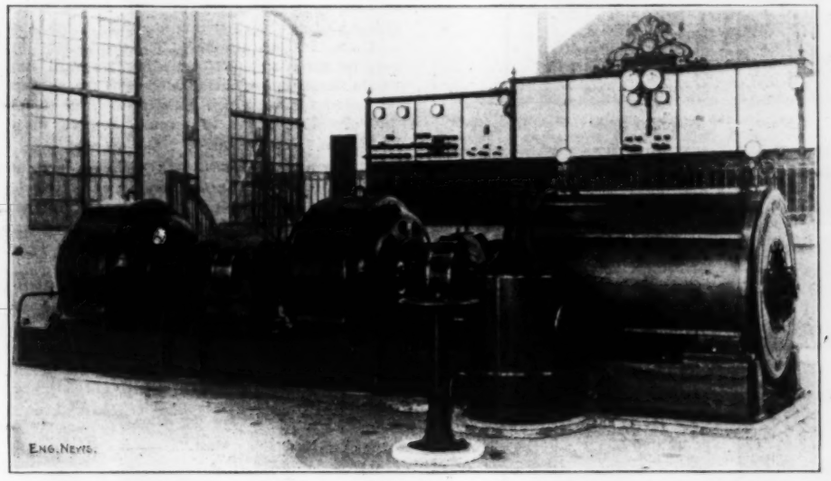


FIG. 2. LOW-PRESSURE RATEAU TURBINE OPERATED BY EXHAUST STEAM.

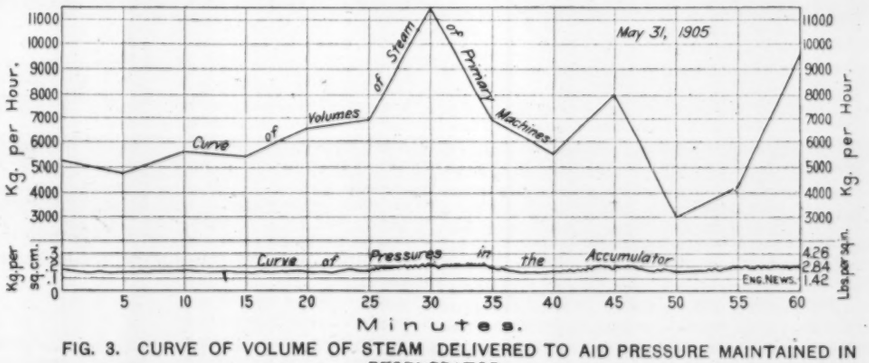


FIG. 3. CURVE OF VOLUME OF STEAM DELIVERED TO AID PRESSURE MAINTAINED IN REGENERATOR.

A UNIVERSAL DICTIONARY OF MECHANICAL DRAWING.*

By George H. Follows.†

Part V. Sectioning and Sectional Views.

60. SECTIONING: This is a subject that has a rather interesting historical side. Time has made such sweeping changes in the work of the draftsman—due largely to the invention of blue printing and other copying processes—that we have come almost to forget what a drawing of a generation ago was like, and to entirely lose sight of the influence that some of the old-fashioned methods have had upon the drawing of to-day.

61. Before the introduction of blue printing, the "original" drawing was used by all concerned, and was quite an elaborate and expensive affair compared with the shop print of to-day. In the first place it was made on the best and toughest of paper—it *had* to be to stand the wear and tear; then it was inked in with genuine "India" ink rubbed down from the stick; and if it was a drawing of a standard machine it was shaded with the brush and symbolically colored according to the

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†Of the Engineering Department, Westinghouse Electric & Manufacturing Co., The Park View, East Liberty, Pa.

materials of construction. Then before being put into service it was carefully mounted on a suitable board or cloth and given a coat of shellac to preserve it and make it washable. Such was the work of some of the "old school" draftsmen of twenty-five or thirty years ago—very different from the "product" of the modern drafting room.

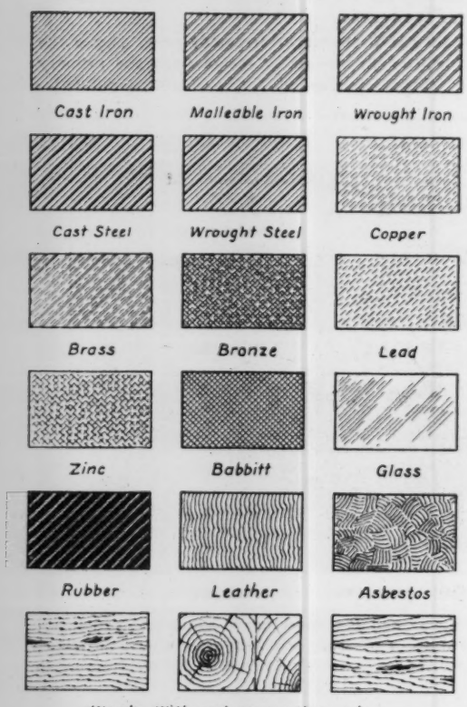

Fig. 14. Standard Section Lining for Different Materials.
(Reproduced from Reinhardt's "Technic of Mechanical Drafting.

62. In the use of the brush, for shading and for the coloring of surfaces in section, the old time practice had a decided influence on the line sectionings that prevail to-day. When colors were used, cast iron in section was represented by an even wash of grey, wrought iron by Prussian blue, steel by a thin wash of crimson lake; while brass was tinted gamboge yellow, and wood was imitated in both color and graining as closely as the draftsman knew how.

63. With the general adoption of tracings and blue prints, shading with the brush died a natural death, giving way first to line shading, later to shaded outlines, and finally to no shading at all; and color sectioning was superseded by line sectioning as used to-day.

64. Now it is necessary to bear in mind that when the custom was to represent materials by symbolical colors, there were comparatively few constructive materials in use, compared, that is, with the far greater number and variety employed at the present time. To a very large extent cast iron was then simply cast iron, brass was brass, and steel—what little there was of it—was not used in so many grades as to call for a special set of symbols. Thus a few colors were able to represent the materials in common use with sufficient precision. So also when the advent of the blue print called for "lines" in place of "colors," it was not necessary to have any great number of symbolical "sectionings." Cast iron was satisfactorily represented by evenly spaced parallel lines; wrought iron by alternate thick and thin ones; steel by lines in pairs; brass by alternate full and broken lines, and so on. But in later years the additional number of materials employed has called for some ingenuity in devising sufficiently distinctive sectionings; the writer has seen authorized varieties that were not only quite difficult to produce, but painfully uncomfortable to look at.

65. Fig. 14 shows a set of sectionings such as are used to-day in many drafting rooms. It is a good set* and would be fairly satisfactory were it not that materials are now made in so many different grades, and there are so many alloys, and so many new materials—or old ones modified to suit special kinds of service—that it is no longer possible to represent materials precisely by symbolical sectionings.

66. Again, many of the big manufacturing concerns now have their own "official specifications" for the materials they purchase or produce, such specifications covering both the chemical and the physical requirements. On drawings, therefore, it is becoming customary to call for a material either by full descriptive name or by general name and specification number. Thus a bearing metal may be specified as "No. 6 Alloy"; a steel may be "Axle Steel, Specification No. 6140"; a shaft may be called for as "Open hearth steel, oil tempered and annealed"; an aluminum bronze may be known as "No. 15 Bronze"; and so on.

67. Symbolical sectioning then is no longer sufficient in itself as an indication of material.

68. These considerations induced the writer some five years ago to suggest that sectionings be used to indicate not individual materials but classes of material.

69. Fig. 15 gives a suggested classification and a set of suitable sectionings. Briefly the classification is well expressed by the meanings of the sectionings themselves; thus:

No. 1 means "Metal."
No. 2 " "Poured into place hot."
No. 3 " "Poured into place cold."
No. 4 " "Fibrous or flexible."
No. 5 " "Flexible insulation."
No. 6 " "Wood."
No. 7 " "Stone class."
No. 8 " "Liquid."
No. 9 " "Special."

70. For the principle involved in this classification three distinct advantages are claimed:

First: There are so few sectionings required that it is not necessary to use any that are at all difficult to produce.

Second: Each sectioning has a constant and definite meaning.

Third: A change of material after a drawing has been made does not necessitate the erasing of a sectioning; unless the new material belongs to a different class, which will very rarely be the case.

71. What may almost be considered as a fourth advantage is illustrated in Fig. 16, where by suitably varying the thickness and spacing of the "metal" sectioning lines the different surfaces show good contrast. This contrast, or "color effect" is fully equal to anything that can be obtained by individual sectionings and is produced with decidedly less trouble.

72. It is at once apparent that the sectioning for a given individual material will not always be the same, but may change with its function or with the method of preparing it. Thus, a separately finished piece of babbitt metal will have sectioning No. 1; the same material poured into place hot will have No. 2. So also with a piece of fiber: Used as insulation (whatever other function it may fill) it will have sectioning No. 5; used mechanically—and not as insulation—it will have No. 4 sectioning. In each case the sectioning points definitely to the class of material, not to the material itself.

*For this set the author is indebted to "Reinhardt's Technic of Mechanical Drafting," in which excellent work the reader may find a comprehensive treatment of some sixty-five very effective individual sectionings.

73. If, as is sometimes the case, it is desirable to specify the individual material in immediate connection with the surface in section, it is easily done by writing the name of the material on the face of the section, or near it; as in Fig. 16.

74. SECTIONAL VIEWS.—The grouping of sectional views is governed by the laws of projection as stated in Part 4, and needs no special comment.

75. When there is a set of sections on parallel planes, the views should be arranged in the reverse order of the planes; and the cutting lines should be so lettered that the views will fall in alphabetical order reading away from the view from which they were projected.

76. In the lettering of cutting lines the planes should be considered as cutting either downward or from left to right and the lines lettered accordingly. Vertical cutting lines will then be lettered AB, CD, etc., reading downwards; horizontal ones AB, CD, etc., reading from left to right; and if inclined, it will generally be easy to decide whether the line should be considered as inclined to the vertical or to the horizontal, and this will determine how the lettering should read.

77. BROKEN SECTIONS.—It is sometimes desirable to make a single sectional view answer the purpose of two or more, by using a bent, or "broken" cutting line instead of a true cutting plane. This kind of section is best indicated by the use of an additional letter at each bend or break in the cutting line and should always be lettered in this manner, as ABC, instead of simply AB; this will distinguish it at a glance from a true section, and will make the drawing just so much easier to read.

PERCOLATION TESTS OF CONCRETE BLOCKS UNDER HIGH HEADS OF WATER AT THE BOROSSA DAM, SOUTH AUSTRALIA.

During the building of the arched concrete dam* at Borossa, South Australia, tests were made to determine the permeability of the concrete under high heads of water. The results of these tests are given by Mr. Alex. B. Moncrieff, M. Am. Soc. C. E., M. Inst. C. E., Engineer in Chief of South Australia, Adelaide, S. A., in Volume XIII of the Transactions of the Association of Civil Engineers of Cornell University, from which the following data have been abstracted.

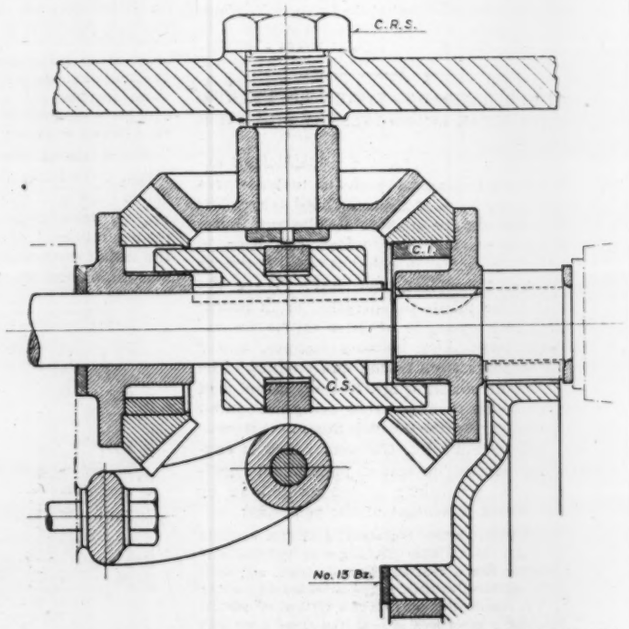

FIG. 16. ILLUSTRATION OF EFFECT OF VARYING WIDTHS OF SECTION LINING FOR DIFFERENT MATERIALS OF THE SAME CLASS.

The aggregate was broken sandstone, ⅛ to

*The dam was described in Engineering News for April 7, 1904.

PERCOLATION TESTS OF CONCRETE BLOCKS SUBJECTED TO HYDROSTATIC PRESSURE.

No. of Block	—Proportions of Ingredients.— Cement.	Sand.	Aggregate (¼" to 2" with varying voids.)	Water used per cu. yd. in manufacture of block, gallons.	Excess Mortar, per cent.	—Head of Water = 100 Ft.— Age of bl'k when first put under pressure, weeks.	Interval bet. turning on of water and appearance on surface, minutes.	Measurable quantity of water passing through block, pints.
1	1	1.84	5.26	28	5	11	(Unreliable)	(Unreliable)
2	1	1.84	5.26	26	5	11	34	¾ in 7 weeks
3	1	1.50	4.63	27	5	10	18	½ in 4 weeks
4	1	2.00	4.50	27	15	10	14	1¼ in 2 weeks
5	1	1.75	4.13	28	15	9	12	27 in 7 weeks
6	1	1.50	4.12	27	10	8	35	⅛ in 2 weeks
7	1	1.50	3.90	24	12½	6	28	¼ in 2 weeks
8	1	1.50	3.70	23	15	5	30	⅛ in 2 weeks

size, with sizes mixed so as to have 35% voids. The sand was a mixture of natural sand and stone dust (⅛-in. and less in size) in about equal proportions.

The combined sands were washed in vertical cylindrical sand washers, by forcing water under a 70-lb. pressure through perforated pipes in the bottom of the washer, and agitating with steam driven revolving paddles.

The concrete was mixed in ½-cu. yd. batches in a Messent concrete mixer which was given 14 revolutions.

Six cubes, 2 × 2 × 2 ft., were made. The article continues as follows:

In the center of each block a T-piece pipe was embedded, and a pressure equal 100 ft. of water was applied through a ½-in. diameter pipe from the top of an adjacent cliff. To prevent the ends of the T-piece becoming blocked with mortar, it was bound around with hemp and small rope to form a bulb about 5 ins. in diameter.

Tests have been made with a view to ascertaining the crushing strength of the concrete and also the rock of which it was composed. The ultimate average crushing strength of the former at the age of four months was found to be 2.15 tons per sq. in., and that of the stone forming the aggregate at 3¼ tons per sq. in.

Tests were also made of the tensile strength of the following sands at the age of 28 days when combined with cement, at a ratio of 3 sand to 1 cement; and the result compared with that of standard sand from the River Torrens were as follows:

	Lbs. per sq. in.
Standard sand, River Torrens	348
River Para sand	424
Mixture of half River Para sand and half rock-crusher sand	460
Dust from rock crusher	192

The accompanying table summarizes the percolation tests.

Blocks Nos. 3 and 6 contained the same strength of mortar as that used in the concrete for the main dam, but with less mortar. The mortar of blocks Nos. 1 to 6 was made with river sand, but for blocks Nos. 6 and 7 equal parts river sand and rock-crusher sand were used.

In all cases, occasional damp spots showed on the outside of the blocks after the 2 to 7 weeks' test of percolation.

At the end of 80 weeks the same blocks were subjected to a 200-ft. head of water, but the percolation was not measured, as in each block the effect closely resembled the results obtained from the head of 100 ft.

YELLOW FEVER AT NEW ORLEANS, up to 6 p. m., Aug. 8, had caused 117 deaths, and there had then been a total of 625 cases, with 114 points of infection. At the request of local and State authorities and business organizations the U. S. Marine Hospital Service, under Surgeon General Wyman, took charge of the outbreak on Aug. 6, with Surgeon J. H. White in direct local charge. Subscriptions are being received to raise a fund of $250,000 to fight the spread of the disease. Dr. White has examined the local ordinances, including the one relating to mosquitos, and states that they give ample legal authority for dealing with the outbreak. There are a few scattered cases outside of New Orleans, mostly in Louisiana, but notwithstanding the small number of these cases many of the outside communities are panic stricken, as is shown by such action as refusing to receive hardware from New Orleans. It is reported that Mayor Behrman has accepted the proffered aid of Sir Alfred Jones, President of the Liverpool School of Tropical Medicine, and as a result there will sail on Aug. 12 Major Ronald Ross and Dr. Robert Boyce, of that school. An abstract of an article by Major Ross appears elsewhere in this issue, and comments on yellow fever and malaria may be found in our editorial columns.

TYPHOID FEVER AT WASHINGTON, D. C., has become markedly prevalent. On Aug. 7 there were 166 cases on record, 46 of which had been reported during the six preceding days. The health officers have advised citizens to boil drinking water. It is a pleasure to announce that the new sand filtration plant is nearing completion.

A PRIZE COMPETITION for investigations on the subject of high-speed trucks for railway cars has been in-

UNIVERSAL DRAWING DICTIONARY

The sections given here are not for individual materials, but are for indicating the broad classes of the materials used in machine construction.

The thickness and spacing of the lines should be varied to suit the size of the surface and the character of the drawing.

1: METAL: — *For all metals except when they belong to Class 2.*

2: POURED INTO PLACE HOT: — *For materials that are to be melted and poured into place hot, and that set solid when cool. Babbitt metal and similar alloys, lead, solder, sulphur, wax, paraffin and pitch are examples. Materials that are to be softened by heat and squeezed into place should also have this sectioning.*

3: POURED INTO PLACE COLD: — *This will generally mean some kind of cement, as Portland and other hydraulic cements, plaster of Paris, concrete, mortar. It should also be used for pastey or plastic materials, as tallow, putty, etc. when squeezed into place without being heated.*

4: FIBROUS OR FLEXIBLE: — *For such materials as leather, leatheroid, rawhide, fibre, asbestos, felt, rubber, gutta-percha, ivory, bone, etc.*

5: FLEXIBLE INSULATION: — (For thin pieces / For thick pieces) — *For use especially in connection with electrical apparatus. Materials that would mechanically belong to the preceding class should have this sectioning when they are used for insulating purposes. Mica, paper, cloth, fibre, fullerboard and rubber are examples.*

6: WOOD: — *It is often desirable to imitate the grain of wood whether in section or not.*

7: STONE CLASS: — *This sectioning should be used for materials that belong to the same general class as stone. Marble, slate, soapstone, brick, terra-cotta, porcelain and glass are examples.*

8: LIQUID: — *For all liquids that do not set solid at ordinary temperatures, as oil, water, alcohol, glycerine, turpentine, etc.*

9: SPECIAL: — *For materials that are entirely special, or that do not belong to one of the above classes.*

FIG. 15. STANDARD SECTIONING FOR CLASSES OF MATERIALS.

stituted by the Verein Deutscher Maschinen-Ingenieure. The sum of 8,000 marks ($2,000) has been set aside for prizes. The investigations are to concern themselves with the question of smooth running at high speed. It is expected that they will demonstrate the requirements which trucks must meet in order to minimize the motions of the car body, both the lateral and vertical motions of its center of gravity and the angular swing of the car about its three principal axes; as well as the requirements for minimum disturbance on curved track. The principles deduced are to be applied in sketch designs. The subjects of track maintenance, and of the various factors influencing the riding of cars, are also to be discussed. Competing essays must be written in German, and are to be delivered by 12 noon of Jan. 2, 1907, at the office of the Verein Deutscher Maschinen-Ingenieure, Lindenstrasse 80, Berlin, S. W., Germany. A copy of the conditions of the competition may be obtained at that address.

THE HOBOKEN PASSENGER TERMINAL of the Delaware, Lackawanna & Western Ry., on the west bank of the Hudson River, opposite New York City, was destroyed by fire in the night of Aug. 7 to Aug. 8. The buildings destroyed include a passenger station, a train shed, a new immigrant building, ferry-slips, and a ferry-boat. A loss to the railway company of several hundred thousand dollars resulted. No fatalities occurred, although many passengers were in the station and in waiting trains. The suburban service of the railway was badly deranged for a day, and through service was somewhat delayed, although arrangements were soon made to convert part of the terminal yard into a station. One of the ferry-slips was in service again twelve hours after the fire, and another five hours later. The structures destroyed were all old buildings of wood frame construction. They were to have been soon replaced by a new terminal of fireproof construction, work on which will now be taken in hand at once. The fire was most serious in the danger which it threatened to the Hoboken water front, including the piers and yards of the Hamburg-American line and the North German Lloyd, lying immediately to the north. By strenuous fire-fighting it was confined to the railway terminal and ferry.

A DRY GOODS STORE COLLAPSED in Albany, N. Y., on Tuesday, Aug. 8. Many persons were killed and injured. The number of killed will probably be in the neighborhood of twenty-five, but may exceed that number. The fall occurred about 9 a. m. The store is about 60 ft. wide by 200 ft. deep, in the middle of a block, and was undergoing alterations. The collapse was due directly to the building operations, and was probably caused by a column or footing failure in a portion of the basement, where excavation for a sub-cellar was in progress.

TESTS OF HEAT INSULATORS for conductivity were made recently in England by R. S. Hutton and J. R. Beard. The substances were tested in granular form; when in large pieces, such as brick, they were crushed to proper size. Conducting power was measured at temperatures up to 100° C., with the following results:

Substance.	Conductivity.
Sand (white Calais)	0.00060
Carborundum	0.00050
Firebrick	0.00028
Lime	0.00029
Magnesia (fused)	0.00047
Magnesia (light calcined)	0.00016
Infusorial earth	0.00013

The tests were reported in a paper read recently before the Faraday Society at London.

PERSONALS.

Mr. B. B. Smith has resigned his position as Engineering Aid in the U. S. Reclamation Service.

Mr. J. A. Hunter has been appointed Chief of the Bureau of Highways of Philadelphia, Pa.

Mr. Sam J. Mills has been appointed Assistant General Manager of MacArthur Bros. Co., Contractors, with headquarters at Chicago, Ill.

Mr. W. M. Probasco has resigned as Assistant to the President of the McGraw Publishing Co., to become Vice-President of the Search Light Publishing Co.

Mr. Weston Small, of Sioux City, Ia., has received an appointment as Engineer in the U. S. Reclamation Service and will report for duty at Yuma, Ariz.

Messrs. William H. Burr, Frederic P. Stearns and John R. Freeman have been appointed Consulting Engineers to the Board of Water Supply of New York City.

Mr. Joseph Herbst, for some years Assistant Superintendent of the electric light plant at Wapakoneta, O., is now Superintendent of Water-Works and Electric Light Plant at Versailles, O.

Mr. W. E. Miller has resigned his position in the Engineering Department of the Pennsylvania Lines west of Pittsburg and has accepted a position as Roadmaster for the Northwestern Elevated R. R., of Chicago.

Mr. W. A. Doane, M. Am. Soc. C. E., has resigned as City Engineer of Meadville, Pa., to accept a position as Division Engineer on the South & Western Ry., with headquarters for the present at Bristol, Va.-Tenn.

Mr. J. C. McClure has been appointed Engineer Maintenance-of-Way of the Arizona & Colorado R. R., the Cananea, Yaqui River & Pacific R. R., and the Arizona Eastern R. R., with headquarters at Tucson, Ariz.

Mr. John C. Fitterer has received an appointment as Assistant Engineer in the U. S. Reclamation Service, and has been assigned to duty under Mr. M. C. Hinderlider as Field Assistant, with headquarters at Denver, Colo.

Mr. Chas. H. Lee, of California, has been appointed Hydrographic Aid in the U. S. Geological Survey, and will report for duty to Mr. J. B. Lippincott, Supervising Engineer, U. S. Reclamation Service, Los Angeles, Cal.

Mr. F. F. Harrington, C. E., has resigned his position as Assistant Engineer to the Chief Engineer of the New York Central & Hudson River R. R. Company, to become Engineer of Bridges of the Tidewater Ry. Co., Haddington Bldg., Norfolk, Va.

Mr. Frederick P. Beach, of California, who has been temporarily employed as Field Assistant with Mr. O. H. Ensign, electrical expert for the Pacific Coast, has received an appointment as Mechanical Engineer in the U. S. Reclamation Service and will remain with Mr. Ensign.

Brigadier-General Peter C. Haines and Mr. Benjamin M. Harrod, members of the Isthmian Canal Commission, have returned to Washington from the Isthmus, and are preparing the physical data collected by them to be placed before the Board of Consulting Engineers when it meets in that city about Sept. 1.

Mr. Paul J. Challen, Assistant to the General Manager of the Green Fuel Economizer Co., has recently accepted a position with The Stirling Co. at their Philadelphia office. Mr. Challen has been connected with the Green Fuel Economizer Co., as designing and constructing engineer, since they started manufacturing in this country 13 years ago.

Messrs. Charles Mebus, of Philadelphia, L. E. Chapin, of Pittsburg, and Robert S. Raithbun, of Allentown, have been selected by State Health Commissioner Dixon, of Pennsylvania, as special engineers to make inspections of proposed extensions to sewer systems in that State. This work eventually will be done by a regular corps of sanitary engineers appointed by Commissioner Dixon, who will make a thorough inspection of sewer systems before permits will be granted for the extension of sewer systems.

Mr. A. F. Woolley, M. Am. Soc. C. E., has accepted the position of Chief Engineer and Superintendent for the District Construction Co., Washington, D. C., succeeding Mr. Ottomor Strange, who has gone to Columbus, O., to take charge of the construction of a large filtration plant for that city. The District Construction Co. is now constructing the Connecticut Ave. Bridge at Washington, D. C., a description of which was given in the Engineering News, of June 1, 1905. Mr. Woolley has recently returned from the Republic of Panama, where he was in charge of the design and construction of terminals, harbors and railways for an American company.

Obituary.

Henry Hodge, a leading contractor of Northern New York, died recently at his home in Watertown, N. Y., aged 69 years.

Winfield S. Wilson, President of the Philadelphia, Germantown & Norristown R. R., died Aug. 4, at Norristown, Pa., aged 80.

A. G. Livingston, of St. Louis, Mo., a Division Engineer on the Panama Canal, stationed at Culebra, died recently at Panama of yellow fever.

Andrew Fletcher, of the W. & A. Fletcher Co., which has engineering works in Hoboken, N. J., died Aug. 8 at his country home at Bernardsville.

Richard C. W. Wadsworth, Secretary to Dr. Lederle when the latter was Health Commissioner of New York City, and later associated with him at the Lederle laboratories, died Aug. 3, at Roosevelt Hospital, New York, from typhoid fever. Mr. Wadsworth is said to have contracted the disease while applying himself to study of how to combat it.

Gen. Roy Stone, formerly U. S. Special Agent and Engineer for Road Inquiry of the Department of Agriculture, died Aug. 6, at Mendham, N. J., after a long illness. General Stone served with distinction during the Civil War, being brevetted a Brigadier-General by President Lincoln, for gallant services during the Peninsular Campaign, and particularly at Gettysburg. In the Spanish War General Stone served on General Miles' staff as Brigadier-General and Chief of Engineers. As an engineer he was connected with many important works, including the blowing up of Hell Gate and the removal of the bars in New York, but he is chiefly known as one of the pioneers in the movement for good roads, and it is largely due to his efforts that New Jersey possesses so many fine thoroughfares. He also advocated the use of steel in the building of wagon ways. General Stone was the projector and promoter of the New York & Long Island Ry., under 42d St. and the East River, New York to Long Island City, and was also the promoter of an underground railway from Bergen, N. J., under Jersey City and Union Sq. to Brooklyn. He was the author of a book entitled "New Roads and Road Laws," which was published in 1894. General Stone was born 69 years ago in Steuben County, New York, and was graduated from Union College just previous to the commencement of the Civil War.

ENGINEERING SOCIETIES.

COMING MEETINGS.

NATIONAL IRRIGATION CONGRESS.
Annual convention, Portland, Ore., Aug. 21-4. Secretary, Tom Richardson, Portland, Ore.

LEAGUE OF AMERICAN MUNICIPALITIES.
August 23 to 25. Annual meeting at Toledo, O. Secy., J. McVicar, Des Moines, Ia.

INTERNATIONAL ASSOCIATION OF MUNICIPAL ELECTRICIANS.
August 23 to 25, 1905. Annual convention at Erie, Pa. Secy., F. P. Foster, Corning, N. Y.

AMERICAN PUBLIC WORKS ASSOCIATION.
Chattanooga meeting postponed until further notice because of yellow fever quarantines. Secy., W. H. Flint, Chattanooga, Tenn.

AMERICAN SOCIETY OF MUNICIPAL IMPROVEMENTS.
Sept. 5 to 7. Annual meeting at Montreal, Canada. Secy., Geo. W. Tillson, Brooklyn, N. Y.

ROADMASTERS & MAINTENANCE OF WAY ASSOCIATION.
Sept. 12, 14. Annual meeting at Niagara Falls, N. Y. Secy., C. E. Jones, C., B. & Q. Ry., Beardstown, Ill.

NEW ENGLAND WATER-WORKS ASSOCIATION.
Sept. 13 to 16. Annual convention at New York, N. Y. Secy., Willard Kent, Narragansett Pier, R. I.

AMERICAN ELECTROCHEMICAL SOCIETY.
Sept. 18 to 20. Annual meeting at Bethlehem, Pa. Secy., S. S. Sadtler, 39 S. 10th St., Philadelphia, Pa.

AMERICAN PUBLIC HEALTH ASSOCIATION.
Sept. 25 to 29. Annual meeting at Boston, Mass. Secy., Dr. Chas. O. Probst, Columbus, O.

AMERICAN STREET RAILWAY ASSOCIATION.
Sept. 25-30. Annual convention at Philadelphia, Pa. Secy., T. C. Penington, 2020 State St., Chicago, Ill.

ASSOCIATION OF RAILWAY SUPERINTENDENTS OF BRIDGES AND BUILDINGS.
Oct. 17. Annual meeting at Pittsburg, Pa. Secy., R. F. Patterson, Concord, N. H.

MAINTENANCE OF WAY MASTER PAINTERS' ASSOCIATION.
Nov. 13, 14. Annual meeting at Cincinnati, O. Secy., H. J. Schnell, 100 William St., New York, N. Y.

RAILWAY SIGNAL ASSOCIATION.—The next meeting of this association will be held at the Great Northern Hotel, Chicago, Ill., on Tuesday, Sept. 12. A morning and an afternoon session are contemplated, beginning at 10.30 and 2.30 respectively. The subjects to be discussed are: Definitions and Nomenclature (a further report of standing committee No. 9), Standard Specifications for Mechanical Interlocking and Material for Construction Work (a full report of committee No. 8), and What is the best method for Adjustment of Cost of Maintenance, Operation, and Inspection, of Joint Interlocking Plant. Secy., H. S. Balliet, Grand Central Station, New York, N. Y.

AMERICAN PUBLIC HEALTH ASSOCIATION.—A preliminary program of the 33d annual meeting of this association has just been issued. The meeting is at Boston, Mass., beginning Monday, Sept. 25, and continuing for five day. The list of subjects to be discussed includes eleven main heads: 1, Methods of securing more uniform instruction and co-operation in health work; 2, Water (Use of copper sulphate, Shellfish and disease, water purification and protection); 3, Tropical hygiene; 4, Diseases common to man and animals; 5, Military hygiene; 6, Quarantine methods; 7, Recent researches in small-pox; 8, Modes of Dissemination and rational quarantine in epidemic cerebro-spinal meningitis; 9, Disinfection; 10, Sewage and waste disposal; 11, Car sanitation.

Details as to the arrangement of sessions, transportation and hotel facilities, etc., will be announced in a later circular. Secy., Dr. C. O. Probst, Columbus, O.

IRON AND STEEL INSTITUTE.—The autumn meeting of the Iron and Steel Institute (Great Britain) will be held at Sheffield, England, on Sept. 26 to 29, 1905. The following papers are scheduled for reading: "The Metallurgical Department of Sheffield University," by Prof. J. O. Arnold, of Sheffield; The Thermal Transformation of Carbon Steels," by Prof. J. O. Arnold and A. McWilliam, of Sheffield; "The Nature of Troostite," by C. Benedicks, of Upsala, Sweden; "The Occurrence of Copper, Cobalt, and Nickel in American Pig Irons," by Prof. E. D. Campbell, of Ann Arbor, Mich., U. S. A.; "Piping in Steel Ingots," by J. E. Fletcher, of Sheffield; "Steel for Motor-Car Construction," by L. Guillet, of Paris, France; "On the Presence of Greenish-Colored Markings in the Fractured Surface of Testpieces," by Capt. H. G. Howarth, of Sheffield; "Overheated Steel," by A. W. Richards ad J. E. Stead; "Segregation in Steel Ingots," by B. Talbot, of Middlesbrough; "A Manipulator for Steel Bars," by D. Upton, of Jarrow; "Machinery for Breaking Pig Iron," by Cecil Walton, of Whitehaven; "The Influence of Carbon on Nickel and Iron," by G. B. Waterhouse, of New York, N. Y., U. S. A. Secy., B. H. Brough, 28 Victoria St., London, S. W.